CONNECTICUT RIVER FLOOD CONTROL

WEST WARREN

LOCAL PROTECTION

QUABOAG RIVER MASSACHUSETTS

DETAILED PROJECT REPORT

JULY 1961

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QUABOAG RIVER, WARREN, MASSACHUSETTS

DETAILED PROJECT REPORT

U. S. ARMY ENGINEER DIVISION, NEW ENGLAND

CORPS OF ENGINEERS WALTHAM, MASS.

JULY 1961

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WEST WARREN

LOCAL PROTECTION PROJECT

QUABOAG RIVER - CHICOPEE RIVER BASIN

WARREN, MASSACHUSETTS

JULY 1961

A. PERTINENT DATA

1. Purpose	Overbank flood control of Quaboag River.
2. Location	Channel improvement upstream and downstream of the existing dam at West Warren, Worcester County, Massachusetts.
3. Type of Improvement	Dike, flood walls, sluice gates, channel excavation, rock slope protection, reconstruction of bridge pier footings, and the removal and modification of two existing bridges.
4. Hydrology	
Maximum flood of record	8,300 c.f.s.
Project design flood	11,000 c.f.s.
Drainage area	210 sq. miles
5. Dike	
Type	Earth filled with rockfill protection on river side, topsoil and seed on landward side
Length	390 feet
Top width	12 feet
Elevation of top	544.5 feet
Ties	Concrete flood wall structure to existing building at one end; existing grade at other end
Side slopes	1 vertical on 2 horizontal

6. Walls

Types	Reinforced concrete T-wall Interior concrete buttress walls
Locations	
T-wall	East of Bldg #7
Buttress walls	North wall interior Bldg #7 North wall interior Bldg #11
Lengths	
T-wall	120 feet
Buttress walls	350 feet, Bldg #7 70 feet, Bldg #11
Heights	
T-wall	26.5 feet
Buttress walls	5 to 7 feet, Bldg #7 8 feet, Bldg #11

7. Sluice Gates

Type	Manually operated cast iron
Location	Flood wall, east of Bldg #7
Number	Two each
Size	4 feet wide x 5 feet high

8. Bridge Pier Footing Replacement

Type	Concrete
Location	North piers of the South St. Highway bridge on the right bank of the Quaboag River opposite Bldg #11
Length	38 feet
Height	25 feet
Width	10'-6" top 14' bottom

9. Removal of Existing 3-Span Stone Arch Bridge — To be accomplished by local interest

10. Removal and Replacement of Utility Bridge — To be accomplished by local interest

11. Rock Slope Protection

Slope	1 vertical on 2 horizontal upstream of the South Street bridge piers, 1 vertical on 1.5 horizontal downstream of these piers.
Cover stone	Minimum 1000 lb quarry stone
Bedding stone	12 inches

12. Channel Improvement

Excavation	900 feet
Bottom width	60 feet
Channel paving	700 feet
Paving stone	Minimum 500 lb
Gravel bedding	12 inches

13. Principal Quantities

Stream control	1 Job
Excavation - unclassified	15,100 c.y.
Rock excavation	1,000 "
Impervious earth fill	2,400 "
Gravel bedding	3,700 "
Cover stone (1,000 lb)	2,400 "
Stone paving (500 lb)	1,700 "
Rockfill	1,100 "
Gravel fill	1,300 "
Concrete work	1,020 "
Sheet piling	2,600 s.f.
Underpin existing South Street bridge	1 Job
Drainage facilities	1 Job

14. Cost Estimates

First Costs:

Federal	$300,000
Non-Federal	45,000
Total	345,000

Annual Costs:

Federal	$ 10,900
Non-Federal	2,500
Total	13,400

15. Benefits

Average Annual Benefits	$ 17,400
Benefit-Cost Ratio	1.3 to 1.0

B. PROJECT AUTHORITY

This Detailed Project Report is submitted pursuant to authority contained in Section 205 of the 1948 Flood Control Act, as amended by Section 212 of the Flood Control Act of 1950 and Public Law 685, 2nd Session, 84th Congress, adopted 11 July 1956. Further authority is contained in 1st Indorsement dated 17 May 1960 from the Chief of Engineers in reply to a report dated 25 March 1960 from the Division Engineer, New England Division, subject: "Reconnaissance Report on West Warren Local Protection Project, Quaboag River, Warren, Mass."

C. SCOPE OF DETAILED PROJECT REPORT

1. SCOPE

This Detailed Project Report reviews the overbank flood problem on the Quaboag River, West Warren, Massachusetts. It submits a definite project for overbank flood control by construction of the following improvements: an earth and rockfill dike; concrete flood walls and intake structure; sluice gates; channel improvements including channel deepening, widening, with stone protection along channel banks and bottom; the removal of two existing bridge obstructions; and the reconstruction of existing bridge pier footings to permit channel deepening.

2. TOPOGRAPHIC SURVEYS

A topographic survey of the proposed local protection project on a scale of 1" = 40' and a contour interval of 2 feet was made in November 1958.

3. SUBSURFACE EXPLORATIONS

Geological reconnaissance of the proposed project area has been made. Subsurface explorations were performed during November 1960 and consist of core borings, earth augurs and test pits; the location and description are shown on Plate Nos. 3 and 4.

4. ECONOMIC INVESTIGATIONS

Surveys of flood damages of the West Warren site were made after the floods of 1938 and 1955. These surveys included personal interviews with officials of industrial concerns that suffered damages. A summary of the results of flood damage surveys are shown in Section L, "Flood Damages and Economic Development."

5. REAL ESTATE STUDIES

Field reconnaissance and conferences with local officials were used as a basis for estimates of real estate costs.

6. CONFERENCES WITH LOCAL OFFICIALS

Close liaison has been maintained with Town and State officials and other interested parties. Desires of local interests are described in Section O. Officials of local concerns have been contacted and the plan of protection explained. All have expressed a strong desire for the immediate construction and completion of the proposed project. Local interests have supplied firm statements as to their willingness and ability to participate in the proposed improvement. Formal assurances will be furnished by the Town and the Commonwealth of Massachusetts, through its Water Resources Commission, prior to completion of final design.

D. PRIOR REPORTS

7. INTERIM REPORT

The Interim Report on Review of Survey, Chicopee River Basin, Massachusetts, dated 8 September 1959, approved by the 86th Congress, 2nd Session, House Document No. 434, prepared by the New England Division, contains preliminary information regarding the subject site.

8. RECONNAISSANCE REPORT

In response to requests from local interests and in compliance with ER 1165-2-102, a reconnaissance and damage survey of the record August 1955 overbank flooding on the Quaboag River, West Warren, Massachusetts was made. The report stated that construction of dikes and flood walls and other channel improvements would relieve the situation. The reconnaissance report indicated that the project was economically feasible and within the scope of Public Law 685. It recommended that the New England Division be authorized to prepare a Detailed Project Report. By first indorsement, dated 17 May 1960, the Chief of Engineers authorized preparation of a Detailed Project Report.

E. DESCRIPTION OF AREA

9. GEOGRAPHY

The Quaboag River, with a length of 26 miles and a drainage area of 210 square miles, lies within the Chicopee River basin which is located in central Massachusetts within the confines of Worcester, Franklin, Hampshire, and Hampden counties. The Chicopee River is formed by the confluence of the Ware and Quaboag Rivers, near the community of Three Rivers in the northwest corner of the Town of Palmer, Massachusetts. The confluence of the Chicopee and Connecticut Rivers is located about 18 miles west of Three Rivers. The Town of Warren is located approximately 14 miles upstream from Three Rivers.

10. TOPOGRAPHY

The entire length of the Quaboag River is characterized by broad, gentle sloping valleys. Except in the dairy country of the Brookfields and Spencer and in parts of Monson, the land in the Quaboag basin is generally sandy and contains large boulders, making it unfavorable for agriculture. The West Warren industrial area is located on a flat flood plain on the left side of the Quaboag River. The Boston & Albany Railroad tracks (owned by N.Y.C.R.R.) are on the right bank of the river. Beyond the tracks, the ground rises rapidly to an elevation high above the flood plain.

11. GEOLOGY

In the project area, the Quaboag River flows through several relatively low glaciolacustrine terraces of sand and gravel. Bedrock outcrops are located on the left and right banks of the river, a short distance below the project site.

12. MAIN RIVER

The Quaboag River, rising in the vicinity of Quaboag Pond in the southeast corner of the Town of Brookfield, Mass., flows in a generally westerly direction for about 26 miles to its confluence with the Ware River at the Chicopee River. In the upper part of the basin, the Quaboag River flows through a large swampy river channel. Valley storage in the area of ponds and marshy flat land is very great and during flood periods huge volumes of water are temporarily stored in this natural basin. The stored floodwaters subside gradually at relatively low rates so that the natural topography produces a flood reduction effect similar to that of a dam and reservoir. The middle reach of the Quaboag River has a relatively steep slope but the lower part of the river, downstream of the mouth of Chicopee Brook, is very flat. The total fall of the river is about 300 feet.

The proposed project site in the community of West Warren, Mass. is along the Quaboag River about 14 miles above its mouth. Here, the Quaboag flows from east to west downstream through the site reach, swings in a southerly direction for about 500 feet and then turns to flow away from the project area in a westerly direction. A small brook enters the river on the left side near the downstream end of the proposed improvement. A profile of the river through the proposed project site is shown on Plate No. 4.

At the project site, the Quaboag River flows over a rock masonry dam, under a 3-span stone arch road bridge, under a 2-span pipeline bridge, and under an abandoned cable supported steam pipe extending across the river to underground fuel storage tanks. The two bridges have insufficient waterway areas and restrict large flood flows. The river also flows under the South Street highway bridge. The bridge span, composed of a steel truss, is elevated about 30' above the river by means of steel trestles which rest upon masonry piers. The bridge is sufficiently high so that flood flows are not impeded.

The majority of the buildings in the industrial area located in the flood plain on the left bank of the Quaboag River are owned by West Warren Industries. It leases space to four firms producing textiles, plastic and metal products. Other space is owned outright by two of the occupants. These firms employ approximately 950 people and constitute the core of economic activity in this portion of the basin. In addition to the vital importance of West Warren Industries to the economic life of West Warren, it maintains and operates the pumping facilities for the fire protection of this community.

13. STREAM CHARACTERISTICS

The existing channel bottom slope in the reach of the Quaboag River that flows through the proposed project site, varies from an elevation of about 528 feet at the upstream end, to an elevation of about 510 feet at station 17+0, which is the downstream end of the proposed improvement. The dam crest, located at station 4+50, is 532.7 feet, while the downstream elevation at the foot of the dam is 520.5 feet. These elevations are shown on Plate No. 4 of this report. Cross-sectional dimensions downstream of the dam show the river bottom width varying from about 110 feet at the dam to about 55 feet in the reach between stations 9+0 and 12+0. Existing bank slopes are about 1 on 1.5 throughout except at station 9+0 where the slope up to the existing piers of the South Street bridge are 1 to 1.

The bank fill capacity for the upstream reach of the river is about 3,500 c.f.s., as determined by an existing bank elevation of 537 feet and the hydraulic rating curve of this area. The capacity of the downstream channel is about 4,000 c.f.s., and is governed by an elevation of 526 feet occurring on the left bank of the river just downstream at Building No. 11.

14. MAPS

The Quaboag River and its watershed are shown on Standard quadrangle sheets of the U.S. Geological Survey (Scale 1:31,680) and on standard quadrangle sheets of the Army Map Service (Scale 1:25,000). A map of the Chicopee River basin is shown as Plate No. 1 of this report.

F. CLIMATOLOGY

15. GENERAL

The Quaboag River basin has a modified continental type of climate and it is generally warm to hot in the summer and moderately cold in the winter. On the average, the precipitation is uniformly distributed throughout the year but frequently subjected to short periods of heavy precipitation. The basin, as well as all southern New England, lies in the path of the "prevailing westerlies" and of cyclonic dusturbances that cross the country from the west or southwest and converge on the northeast. These well developed cyclonic storms produce rapid weather changes and act as an important climatic control. The area is also exposed to occasional storms that travel up the Atlantic Seaboard, some of which are of tropical origin and of hurricane intensity. These storms have a high potential for flood-producing rainfall, particularly from August to October.

16. TEMPERATURE

The average annual temperature in the basin is approximately 50°F. Recorded temperature extremes at representative stations within or adjacent to the Quaboag River basin have varied from a maximum of about 100°F. to a minimum of about -20°F. Freezing temperatures have been experienced from the latter part of September until the early part of May. Mean, maximum, and minimum temperatures are shown in Table 1 for long period record stations near the Quaboag River basin.

TABLE 1

MONTHLY TEMPERATURES
(Degrees Fahrenheit)

Elevation	Worcester, Mass.			Westover Field, Mass.		
(ft. m.s.l.)	628			240		
Years of Record	67			18		
Month	Mean	Max.	Min.	Mean	Max.	Min.
January	25.4	69	-18	25.8	65	-21
February	25.5	67	-24	28.1	65	-18
March	34.9	84	-6	36.6	86	-13
April	45.8	91	8	47.5	87	13
May	57.3	92	25	58.1	93	29
June	65.9	98	33	67.7	102	37
July	71.0	102	41	72.5	97	45
August	69.0	99	35	39.9	99	36
September	62.0	100	26	62.1	101	27
October	51.7	89	13	52.8	89	17
November	40.2	81	3	41.7	81	8
December	28.5	67	-17	29.1	64	-15
Annual	48.2	102	-24	49.3	102	-21

17. PRECIPITATION

The mean annual precipitation of approximately 40 inches at West Warren is distributed uniformly throughout the year. In the headwaters of the Chicopee River basin, at Hubbardston, Mass., the maximum and minimum annual precipitation, respectively, for 38 years of record was 61.3 inches in 1938 and 32.8 inches in 1949. At the Ware, Mass. No. 2 climatological station, the maximum and minimum annual precipitation for 27 years of record was 62.8 inches in 1955 and 33.5 inches in 1935, respectively. The mean, maximum, and minimum monthly recorded precipitation for these stations, which are representative of the West Warren area are summarized in Table 2.

TABLE 2

MONTHLY PRECIPITATION RECORD
(In Inches)

	Hubbardston, Mass.			Ware #2, Mass.		
Elevation, (ft. m.s.l.)		1,030			500	
Years of Record		38			27	
Month	Mean	Max.	Min.	Mean	Max.	Min.
January	3.04	6.17	0.61	3.27	6.46	1.00
February	2.53	4.83	1.36	2.84	5.55	0.93
March	3.58	8.89	1.32	3.74	7.52	1.59
April	3.73	7.62	1.01	3.37	6.49	0.71
May	3.59	6.96	1.38	3.76	6.27	0.70
June	4.29	11.82	0.46	4.46	8.29	1.22
July	4.10	8.00	1.07	3.94	10.14	1.21
August	3.47	9.83	0.79	4.14	20.65	0.46
September	4.30	18.28	0.94	4.11	15.70	1.02
October	3.28	8.05	0.13	2.97	9.12	0.81
November	4.04	7.44	0.95	3.73	8.41	0.83
December	3.04	6.83	0.50	3.41	6.11	0.74
Annual	42.99	61.34	32.81	43.74	62.78	33.51

18. SNOWFALL

The average annual snowfall at West Warren is about 50 inches. The water equivalent of the snow cover generally reaches a maximum in March with an average basin depth of 2 or 3 inches with maximum basin averages over 5 inches. The mean annual snowfall recorded at Westover Field at Chicopee Falls, Mass., for 15 years of record, is 50 inches. At Hubbardston, Mass., representative of the Chicopee River basin area, the annual mean, maximum, and minimum snowfall was 54 inches, 98 inches, and 36 inches, respectively, during the period from 1930 to 1955. The mean monthly snowfall for the periods of record at Westover Field and Hubbardston are summarized in Table 3.

TABLE 3

MEAN MONTHLY SNOWFALL
(Average Depth in Inches)

	Westover Field, Mass.	Hubbardston, Mass.
Elevation, (ft., m.s.l.)	240	1,030
Years of Record	11	25
Month		
January	12.5	14.9
February	14.0	14.2
March	8.9	7.6
April	1.7	2.5
May	0	0.1
June	0	0
July	0	0
August	0	0
September	0	0
October	T	0.4
November	2.0	4.2
December	11.2	10.1
Annual	50.3	54.0

19. STORM TYPES

The New England region experiences a number of middle latitude cyclonic storms each year, generally approaching from the southwestern quadrant. They are most common during the winter months but may occur during any season. For example, flood-producing storms occurred over New England in November 1927, March 1936 and July 1938.

Storms of tropical origin, while of infrequent occurrence over New England, may produce extremely intense rainfall. Late summer and early fall is the usual season for tropical storms and hurricanes.

Precipitation records of many years' standing were broken by the hurricane of September 1938, only to be again exceeded by the tropical storm of August 1955. The latter storm produced about 15 inches of rainfall over the Quaboag River basin, which is by far the greatest amount of precipitation ever recorded in the area.

G. RUNOFF AND STREAMFLOW DATA

20. DISCHARGE RECORDS

The U. S. Geological Survey has published records of river stages and streamflow on the Quaboag River at West Brimfield since 1912. West Brimfield is about three miles downstream from the proposed project site. The records are good except those for periods of ice effect, or no gage height record, which are fair. The gaging station has a drainage area of 151 square miles and a continuous record since 1921. The mean, maximum instantaneous, and minimum discharges are 244 c.f.s., 12,800 c.f.s., 6.6 c.f.s., respectively, for a 47-year record (1912-1958).

21. RUNOFF

The annual runoff for the 47 years of record at West Brimfield varied from 104 c.f.s. to 430 c.f.s., with a mean of 244 c.f.s. Table 4 is a summary of the maximum, minimum, and mean monthly runoff (in c.f.s.) for the period of record at West Brimfield.

TABLE 4

AVERAGE MONTHLY RUNOFF (c.f.s.)

Month	Maximum	Minimum	Mean
January	587	49.0	261.7
February	621	71.8	255.3
March	1,399	207	496.6
April	1,352	173	544.9
May	573	108	322.5
June	655	63.5	181.3
July	524	25.7	110.6
August	1,440	12.8	120.6
September	1,369	12.0	128.2
October	607	11.9	108.5
November	693	26.9	169.0
December	600	48.5	225.4
Annual	430	104	244

H. FLOODS OF RECORD

22. NOTABLE FLOODS

The first of two March 1936 floods was caused by a combination of rainfall and snowmelt. The second flood, which was the more destructive, involved only minor amounts of snowmelt but considerably more rainfall over this basin. Also, the second flood followed the first so closely as to occur with nearly saturated ground conditions and nearly bank full river channels. Other notable floods occurring in September 1938 and August 1955 were caused by hurricanes originating in the tropics and following a northerly path along the east coast of the United States.

23. FLOODS OF RECORD

On the basis of meager records, it appears that the Quaboag River basin had not experienced many destructive floods prior to 1936. Damaging floods occurred in May 1854 and October 1869, but as far as can be determined, neither approached the magnitude of the major floods that occurred during and after 1936. Four floods of major proportions have occurred since 1936. Pertinent data on these floods at West Brimfield are listed in Table 5.

TABLE 5

FLOODS OF RECORD

WEST BRIMFIELD, MASS. (USGS GAGE)

Date of Flood	Average Rainfall (inches)	Peak Discharge Observed (c. f. s.)	Gage Height (feet)
12 March 1936	2	2,040	6.8
19 March 1936	5	3,620	8.62
21 Sept 1938	11.5	8,470	11.8
19 Aug 1955	15	12,800	14.79

I. DISCHARGE FREQUENCY

The frequency, or percent chance of occurrence of peak discharge was determined for West Brimfield USGS Gage and interpolated for West Warren by assigning the same computed mean per effective square mile to the 0.7 power skew coefficient and August 1955 flood frequency. A tabulation of discharge frequency at West Warren is shown in Table 6.

TABLE 6

DISCHARGE FREQUENCY

QUABOAG RIVER

West Warren, Mass.

Average Recurrence Interval, Years	Peak Discharge (c.f.s.)
1	900
2	1,200
5	1,600
10	2,200
20	3,000
50	4,600
100	6,400
200	9,000

J. STANDARD PROJECT FLOOD

A standard project flood at West Warren was developed from a standard project storm centered very near West Warren. An analysis of this flood is described more fully in "Northeast Flood Studies Interim Report on Review of Survey, Chicopee River Basin, Massachusetts by the U. S. Army Engineer Division, New England, Corps of Engineers," dated 8 September 1959. Due to the pronounced influence of the ponds and marshy area above Warren upon downstream discharge, a compound storage routing was used to determine the outflow from the natural storage basin. It was found at the time of peak discharge at West Warren, the flood inflow into the ponds and marshy area above Wickaboag Pond outlet went entirely into storage. The standard project flood at West Warren was, therefore, produced entirely by the watershed downstream of the Wickaboag Pond outlet.

K. PROJECT DESIGN FLOOD

The standard project flood, with a peak discharge of 11,000 cubic feet per second, was adopted as the project design flood. Design of the project against this flood which represents a flow 133 percent larger than the flood of record (8,300 cubic feet per second) will insure a reasonably high degree of protection to the industrial properties.

L. FLOOD DAMAGES AND ECONOMIC DEVELOPMENT

24. EXTENT AND CHARACTER OF FLOODING

Nearly nine acres of highly industrialized area, located on the left bank of the Quaboag River in West Warren, are subject to serious flooding. High river flows overtop the river bank adjacent to the left abutment of the dam and flow through the mill buildings and yards. In addition, the channel downstream of the dam is inadequate to contain flood flows and water backs into the buildings adjacent to the river downstream of the dam. In the flood of August 1955 seven industrial buildings were inundated to depths of six feet, and depths of $4\frac{1}{2}$ feet were experienced over the ground. The September 1938 flood produced flooding about two feet less than experienced in 1955. The value of the land and improvements, including seven industrial buildings in the flood area are estimated at $3.5 million. The property is owned by the West Warren Industries, a realty corporation which leases space to four firms producing over $15 million worth of textiles, plastics and metal products annually. These firms employ 950 people, with an annual payroll of over $3 million. The plants in the flood area which provide the economic base for the town, are also a key source of employment for several towns bordering West Warren. Water for the fire protection system for the village of West Warren, population 1,100, is pumped from the Quaboag River by pumps located in the boiler house of West Warren Industries.

25. TRENDS IN DEVELOPMENT

All available space in the buildings in the flood area is currently occupied and being put to a high type of use. The tenant firms enjoy a stable, relatively high level of prosperity. Topography limits physical growth in the area and the character of the present processes makes it unlikely that there will be any higher type of usage of existing facilities. The demand for the products of the industries has remained at a high level, and forecasts indicate that the general level of current activity will prevail in the future.

26. FLOOD DAMAGES

The record flood of August 1955, on the Quaboag River, caused estimated damages of nearly $1 million in the flood area. Seven industrial buildings were inundated to depths of six feet, forcing shutdown of multishift operations for a period of seven to ten days. In addition, the swift floodwaters washed out a service road, inundated a steam plant furnishing steam for heat and industrial processing. Fire pumping facilities were interrupted, jeopardizing the safety of the entire town of West Warren in the event of fire. Flood stages, a few inches higher than those of the August 1955 flood, would enter additional critical space and cause substantial increases in losses.

27. RECURRING LOSSES

A recurrence of the August 1955 flood, the flood of record on the Quaboag River, would cause an estimated loss in West Warren of $1.1 million under present economic conditions. Of this amount, approximately $660,000 loss would result from headwater stages above the dam and $440,000 would result from high tailwater stages. A recurrence of 1938 flood stages would occasion damages of $870,000. Construction of the project would eliminate all losses in a recurrence of either of these floods.

28. AVERAGE ANNUAL LOSSES

Damage-frequency curves using recurring stage-damage, stage-discharge and discharge-frequency relationships, result in estimated average annual losses of $18,700 under present pre-project conditions. Construction of the project would reduce losses to $1,300 annually.

M. EXISTING CORPS OF ENGINEERS FLOOD CONTROL PROJECTS

There are no existing Corps of Engineers flood control projects on the Quaboag or Chicopee Rivers. The three completed projects in the Chicopee basin are Barre Falls Reservoir, Ware Local Protection Works, and Chicopee Local Protection Works. Several proposed
p

projects exist in the Chicopee basin. Conant Brook Reservoir, Three Rivers Local Protection and Chicopee Falls Local Protection were all authorized by Public Law 86-645, 86th Congress, dated 14 July 1960. The West Brookfield Reservoir in Brookfield, which was authorized by the Flood Control Act of 18 August 1941, H. D. 724, 76th Congress, and held inactive due to a lack of economic justification, was subsequently deauthorized by Public Law 86-645. The locations of these projects are shown on Plate No. 1.

N. IMPROVEMENTS BY FEDERAL AND NON-FEDERAL AGENCIES

29. FEDERAL

The Soil Conservation Service, Department of Agriculture, providing technical assistance to the Southern Worcester County and Northwestern Worcester County Soil Conservation Districts under the authority of Public Law 566, Watershed Protection and Flood Prevention Act, has developed a Work Plan for the Upper Quaboag River Watershed. The Work Plan includes five single-purpose floodwater retarding structures, two multiple-purpose floodwater retarding and fish and wildlife improvement structures, one multiple-purpose floodwater retarding and recreational development structure, 818 feet of concrete flood wall, and 2.66 miles of stream channel improvement, all located upstream of West Warren. The Work Plan has been submitted to the Administrator, Soil Conservation Service, for approval. The Work Plan, including its elements, is a result of coordinated studies by both the Soil Conservation Service and the Corps of Engineers and will have no adverse effect on the project at West Warren. (See letter of Comment, Appendix C).

30. NON-FEDERAL

Subsequent to the flood of 1938, the owners of West Warren Industries constructed a flood wall along the right abutment of the existing dam. Approximately 400 feet of railroad track and embankment was replaced by the Boston & Albany Railroad Company.

After the disaster of 1955, West Warren Industries accomplished several improvements. These included a new intake structure, a small stoplog structure between Buildings 7 and 11, and a new concrete wall along the tributary stream that flows behind Building 8. In Building 7, 8000 c.y. of fill was placed under a new concrete floor.

Channel improvements in the Quaboag River, after the flood of August 1955, consisted of removal of accumulated boulders and debris in the vicinity of the oil pipeline bridge and oil storage tanks. This work was accomplished by having a bulldozer work in the river channel, pushing the boulders onto existing bank slopes. No improvements to the area have been made by either State or County.

O. IMPROVEMENTS DESIRED

Several meetings have been held between representatives of the New England Division, Corps of Engineers, and local interests. The citizens and manufacturers of West Warren are very desirous of preventing future losses to the various industries from which they derive their livelihood. They believe that the proposed project as outlined herein will be beneficial to the community, particularly to its economic base, by preventing damages to its principal industries. Local interests have expressed a willingness to fully cooperate on this proposal for the protection of West Warren Industries. They have informed the Corps of Engineers that the Town will comply with local cooperation required by law subject to concurrence by vote of the Town meeting. Prints of letters giving the views of local interests are included in Appendix C of this report.

P. FLOOD PROBLEM AND SOLUTIONS CONSIDERED

31. FLOOD PROBLEM

The Quaboag River is susceptible to floods caused by heavy rains or a combination of heavy rains and melting snow. The 1955 flood of record caused severe damage to the area by inundating most of the buildings that comprise West Warren Industries. A 7-foot head of water built up behind the existing dam, causing floodwaters to overflow both banks of the river. Overbank flooding along the left bank of the river incurred severe damages. High velocities prevented gate closure of the intake structure and resulted in the scouring of portions of this conduit. Floodwaters flowed into Building 7 from all sides, principally from windows and doors, inundating the first floor to a depth of five feet. Fortunately, there were no structural failures to the many building walls. This was perhaps attributable to inundation of plant yards which had the effect of equalizing hydraulic pressures against brick exterior walls. However, on the right bank of the river only slight damage was incurred to the railroad tracks in the form of minor erosions to bank slopes. This in part was attributable to flood walls constructed along the right abutment of the dam after the 1938 flood. During the 1938 flood, waters circled the right embankment of the existing dam, scouring out about 400 feet of railroad track and the north abutment of the dam. Downstream of the dam, the constrictions caused by insufficient channel area and the principal hydraulic obstructions; namely, the oil pipeline bridge, and the 3-span stone arch bridge, caused backwater to flow into the buildings and also to erode the pier footings of the overhead South Street bridge. Further restriction to the flow was induced by several large boulders located downstream of this bridge.

32. SOLUTIONS CONSIDERED

Consideration has been given to all practicable methods of solving the flood problem at West Warren. Realignment of the existing channel is impractical and not economically justified due to physical and topographic features of the area. The cost of diversion floodwaters, or construction of upstream reservoirs, to be effective, was found to exceed the benefits derived.

The solution to the flood problem as set forth in the "Reconnaissance Report" was considered to be the best solution to the problem. Detailed studies revealed, however, that economic justification was available for a higher degree of protection than had been previously contemplated. Another predominant change in the improved plan is the substitution of more extensive channel deepening and the placement of heavy cover stone on the bank slopes in lieu of a concrete wall considered previously for the protection of the oil storage tanks. Detailed studies similarly revealed the need for wall construction at Building 7, both along its north and east sides. In addition, buttress wall construction was found necessary along the north wall of Building 11. Positive protection of Buildings 7 and 11 was found necessary to make effective the contemplated improvements upstream of the dam. Studies indicated that channel deepening was the only positive way of reducing the backwater condition in the vicinity of the South Street bridge and Building 11. However, replacement of the north pier footings of the overhead South Street bridge was found necessary to prevent the undermining of the existing pier by the channel excavation. Attention is invited to Plates Nos. 3 and 5 for plan and typical sections of proposed improvements.

Alternate proposals for the channel widths and depths were thoroughly investigated as were plans for different types of concrete wall construction. The most practical and economically justified solution to the problem resolved to be a combination of controlling streamflows upstream of the existing dam by means of an earth dike and construction of appurtenant concrete walls, and downstream channel excavations and improvements.

Q. PROPOSED IMPROVEMENT

33. GENERAL DESCRIPTION

The proposed plan for flood protection of West Warren Industries is indicated on Plate No. 3. The plan includes the following: a dike and gated flood wall upstream of the existing dam; construction of concrete buttress walls to protect Buildings 7 and 11; removal of the existing stone arch bridge, and removal and replacement of an oil pipeline bridge by a new bridge with adequate channel clearance; rock and earth channel excavation with rock protection along bank slopes and channel bottom; and the replacement of the South Street bridge pier footings along the north bank of the river to permit channel deepening at this critical location.

34. DIKE

The upstream impervious fill dike, to be constructed on the left bank of the river, will be approximately 390 feet long. The dike will have a top elevation of 544.5 feet m.s.l., beginning at grade at its easterly end and terminating in a concrete flood wall at its westerly end. The river side of the dike will have a 12-inch layer of rockfill resting upon 12 inches of gravel covering an impervious fill section. The landward slope will be topsoiled and seeded. Poor soil conditions in the vicinity of the dike necessitate stripping of 6 inches of topsoil. In addition, the porosity of the existing fill material behind the existing stone wall necessitates a 6-foot impervious cutoff trench to prevent seepage. A cross-sectional view of this dike is shown on Plate No. 3.

35. CONCRETE FLOOD WALLS

The proposed plan of improvement includes three flood walls. One flood wall is located upstream of the existing dam, and two separate flood walls are located in Buildings 7 and 11. The walls, as indicated in plan on Plate No. 3, are as follows:

a. The upstream reinforced concrete T-wall (see plate No. 5) is utilized to tie the dike to the east wall of Building 7. The T-wall has a top elevation of 544.5 feet, m.s.l., and is provided with two gated openings 4 feet wide by 5 feet high, which will permit water regulation to the existing intake. The flow through these new sluice gates is channeled to the existing intake gates by means of a 12-foot wide open culvert. This culvert will be approximately 45 feet in length with an invert elevation of 529.0 feet m.s.l. The new stone walls of this channel will utilize rock taken from the demolished stone arch bridge. The foundation material for these walls will be similar to the gravel fill material to be placed in the inclosed area as shown on Plate No. 5. The gravel fill will have a top elevation of 538.0 feet m.s.l., with a slight pitch in grade away from existing walls toward the new culvert. A concrete chamber will be provided at the new gates to accommodate stoplogs to permit repairs or maintenance on the gates. The T-wall has a top of slab elevation at 520 feet, m.s.l. for its entire length in the river. In the last 20-foot length, where the wall ties into the new dike, the wall will step up in four 3-foot increments, placing the top of the slab of the last 5-foot section at elevation 532.0 feet, mean sea level.

b. A second wall is located on the interior of the north wall of Building 7 and follows along its entire length of 350 feet. This concrete buttress wall will reinforce the existing brick exterior wall against the standard project flood of 11,000 c.f.s. Attention is invited to the profile shown on Plate No. 4 of this report. Additional height in the wall is provided to allow for a freeboard zone of from 2 to 2.5 feet. The top elevations of the wall vary from 531.0 m.s.l. at its upstream end (east) to 529.0 m.s.l. at the downstream end (west). The wall is set into fill two feet below the existing floor elevation of 526.6 feet m.s.l. A toe drain and 6-inch perforated pipe will intercept and control seepage, to prevent any structural damage to the floor slab.

c. An interior concrete buttress wall similar to the wall described in paragraph b above is incorporated on the north side of Building 11 to provide protection against the flood profile as shown on Plate No. 4. The wall, with a top elevation of 528.5 m.s.l., will run for a length of 70 feet and will provide a minimum of 2.5 feet of freeboard.

36. CHANNEL IMPROVEMENTS

The major features of channel work will occur downstream of the dam. The work will include deepening and widening of the existing channel, and rock protection along the new bottom and side slopes. A new channel bottom width of 60 feet will be provided between stations 8+00 and 12+50. The channel excavation will have a maximum depth of 2.5 feet below its present bottom. Upstream of station 8+00 the channel bottom width varies from 60 feet to about 110 feet at the foot of the dam. Stone protection along the right bank between the dam and the site of the existing three-span stone arch bridge, is to be placed on a slope of 1 vertical on 2 horizontal. Stone will also be placed at a 1 on 2 slope on the left bank in order to channelize flood flows and to afford protection to the foundation of Building 7. Downstream of the site of the stone arch bridge, owing to space restrictions from existing structures, bank slopes will transition from the 1 on 2 slope to a 1 on 1.5. The transition will be completed in the 100-foot distance between Stations 8+0 and 9+0. Downstream of Station 9+00, the banks will have a 1.0 vertical on 1.5 horizontal slope extending to the limits of the project. Anticipated flood velocities of up to 14 feet per second dictate a minimum cover stone of 1,000 pounds on the bank slopes. Stone paving of the channel bottom will utilize 500-pound minimum weight for a distance of 700 feet between stations 5+50 and 12+50. In the vicinity of stations 15 through 17, there are numerous boulders obstructing channel flow. These boulders will be removed. Three slope projections in this vicinity will also be excavated and provided with a rockfill cover on the new slopes. The details for stone protection are more fully covered in paragraph 40.

37. BRIDGE PIER FOOTING

In order to meet the hydraulic channel requirements, a major construction feature of this project involves the replacement of the existing north pier footings of the South Street bridge with a new concrete footing 38 feet long by 14 feet wide. The height of the proposed concrete footing will be 25 feet with a bottom and top elevation of 508.0 and 533.0 feet m.s.l., respectively. The proposed channel bottom in this area will be at elevation 514 feet m.s.l., whereas the estimated bottom of the existing footing, as established by foundation test pit, is at elevation 518. This required elevation would undermine the existing footings.

The two bridge spans which the existing piers support will be underpinned and braced by temporary trestles during the construction of the new pier footing. All vehicular traffic will be detoured. Steel sheet piling utilized as form work for the new concrete footing is required to stabilize the existing embankment and prevent undermining of the railroad tracks during construction. The cost of this work is currently estimated at $40,000 which is recommended to be considered as a Federal cost. Reference is made to EM 1120-2-101, page 65, paragraph 1-84, h (2), which states:

"(2) Piers - When deepening of channels below footings of existing bridge piers is required, the piers may be reinforced, underpinned or reconstructed by the Federal Government, provided it has been released from any claims for damages."

Prior to start of construction on railroad property, formal assurances from the New York Central Railroad Company to release the Town of Warren from any claims for damages, will be obtained. A letter of intent, indicating that the New York Central Railroad is willing to cooperate with the Town in the Proposed Local Protection Project is included in Appendix C of this report.

38. BRIDGE REMOVAL AND REPLACEMENT

The plan of improvement calls for the removal and modification of three existing bridges which cross the Quaboag River at the project site in order to eliminate the channel obstruction which they present.

Of the three bridges, the three-span stone arch bridge represents the major obstruction to channel velocity. During the 1955 flood, a backwater condition at this bridge contributed to flooding of Building 7. Obstruction was partially due to the snagging effect of the piers, and the restricted area afforded by its arches. This bridge, which was utilized principally as access to the oil storage tanks will be removed and, in accordance with the wishes of local interests, will not be replaced. The removal of the bridge will be accomplished by West Warren Industries, in agreement with the Town. Stone salvaged from the bridge will be made available for use in the construction of the channel walls upstream of the existing structure.

During the August 1955 flood high water flowed over the oil pipeline bridge and the abandoned steam line and support. The oil pipeline bridge is supported by a stone pier at its center while the steam line is hung from a cable with a clear span from bank to bank. Both the pipeline bridge and the steam line will be removed by local interests. A new utility bridge will be constructed by them which will provide sufficient channel area to safely pass the design flow.

39. HYDRAULIC DESIGN

a. General. The water surface profile for the standard project flood (11,000 c.f.s.) is shown on Plate No. 4. The profile was computed by conventional backwater methods as described in EM 1110-2-1409, 7 December 1959, Backwater Curves in River Channels. Starting elevations were taken from discharge rating curves at selected stations developed from recorded high water data. For hydraulic analysis, the project area was divided into two reaches separated by the West Warren Industries dam, which is a hydraulic control at all river stages investigated. A roughness coefficient of 0.035 was assumed throughout the reaches. Additional losses were assumed for the effect of turbulence and impact where considered applicable, particularly in the curved channel section downstream of the dam. An attempt to analyze the hydraulics of the river as experienced during the August 1955 flood proved unsuccessful due to lack of channel information and subsequent changes by local interests.

b. Upstream of dam. The water surface in the project reach upstream of the dam was determined by backwater computations starting from the discharge-rating curve at the dam. The design water surface elevation varies from 541.6 at the dam to 541.8 at Station 0+00, about 450 feet upstream. The backwater computations show that the velocity will range from about 4 to 12 feet per second along the flood wall to about 4 feet per second along the dike. The top of dike and flood wall elevation was selected at 544.5, which will provide a minimum freeboard of 2.7 feet above the standard project flood profile.

c. Downstream of dam. Hydraulic investigation revealed that removal of the primary obstructions, the stone arch and oil pipeline bridges, alone would not sufficiently reduce the water surface profile and must be accompanied by channel deepening and widening, particularly in the vicinity of South Street bridge and Building 11. Local interests have agreed to remove the stone arch and oil pipeline bridges and will replace the latter with a more adequate structure. Studies indicate that the most feasible plan for improvement in the vicinity of South Street bridge will necessitate excavating the channel a maximum of 2.5 feet and reconstructing the existing piers on the right bank. The improved channel will have a minimum bottom width of 60 feet and will be protected by stone riprap. Backwater computations indicate that the velocity will be relatively high between the dam and Station 12+50, varying from 13 to 16 feet per second. Below Station 12+50 the channel has a much larger cross section and the water will decelerate. The channel invert will be provided with stone paving between Stations 5+50 and 12+50. The side slopes between Stations 4+50 and 17+00 will be protected by rock to a minimum height of three feet above the design water surface, including superelevation. The design water surface will be superelevated about 1.5 feet in the curved section between Stations 8+50 and 12+20. A low dike will be required on the left bank between Stations 9+00 and 11+20 to prevent backflow into the mill area. Numerous large boulders will be removed from the channel near Station 5+00 and 15+50 to reduce tailwater effect and improve gravity flow from the interior drainage system. The exposed riverside walls of Buildings 7 and 11 will be reinforced with interior concrete buttress walls to a height of 2.0 to 2.5 feet above the design water surface profile.

d. Channel protection. Hydraulic Design Charts 712-1 and 712-1/1 in the data book of Hydraulic Design Criteria were used as guides in the selection of stone riprap protection for the channel invert and side slopes. In general, the USBR Curve shown on the above chart was used to determine minimum stone weights at critical locations. Slope protection for the dike upstream of the dam will be provided by rockfill. The heel of the flood wall immediately upstream of the dam will be protected from undermining by stone having a minimum weight of 500 pounds. Below the dam, where average velocity will vary from 13 to 16 feet per second, slope protection will be provided by stone having a minimum weight of 1,000 pounds. The channel invert in this reach will be paved with stone having a minimum weight of 500 pounds.

40. GEOLOGY, FOUNDATIONS, EMBANKMENTS AND MATERIALS

a. Site Geology. In the project area the river flows through deposits of glaciolacustrine and outwash sands and gravels which formerly choked the Quaboag Valley and lower reaches of many of the tributary valleys. At present, remnants of these deposits are represented in low, erosional terraces adjacent to the rivers. Downstream of the West Warren Industries Dam, the river bed is filled with cobbles and large boulders. Granitic bedrock outcrops just above river level on the left bank a short distance downstream of the project limits.

Geological reconnaissance of the project and adjacent areas was made in connection with site studies and materials surveys. Subsurface explorations included five (5) foundation drive-sample borings, one (1) foundation hand auger boring and one (1) foundation test pit. Two of the borings were made to determine subsurface conditions at the dike site, two to determine the existing foundation conditions along the right bank downstream of the highway bridge, one to de-determine the nature of foundation materials at the site of the T-wall and one to secure data relative to existing foundation conditions beneath the large mill buildings on the left bank just below the dam. The test pit was dug to determine the nature and condition of footings and foundations of the existing highway bridge over the railroad.

Overburden materials in the project area comprise a conglomeration of fill materials, sands and gravels, glacial till, cobbles and boulders. The fill generally consists of silty fine to gravelly sand and, in some places, contains layers of cinders and fragments of wood, glass, brick, asphalt, concrete, etc. This material caps the low terraces adjacent to the river to depths of 5 or more feet and is present to some extent beneath the floors of Buildings 7 and 11. The sands and gravels which underlie the fill in the low terraces are for the most part variably silty and rather pervious, though thin sections of relatively clean and highly pervious gravels can be expected. Such a pervious section may be present at a depth of 12 feet in the left bank terrace upstream of the dam, upon which the dike will be founded. In the dike area, the ground surface of which is some 4 feet above river level, these terrace deposits and the overlying fill are protected on the river side by a masonry wall. Elsewhere in the project area the riverward slopes are partially covered with dumped material ranging from cobbles to large boulders. It is in and possibly upon these terrace deposits that the piers of the highway bridge and the walls of Buildings 7 and 11 are founded. The terrace deposits and the fill constitute the bulk of materials to be excavated in channel widening operations and in the reconstruction of highway bridge piers. No rock excavation other than boulders and blocks is anticipated. Sands and gravels similar to those in the terraces can be expected to be present in the river channel throughout the project. Downstream of the dam these materials are obscured by a pavement of cobbles and boulders from 1 to 5 cubic yards in volume and which constitute the bulk of channel deepening excavations. Underlying the sands and gravels throughout the project area is a compact, sandy glacial till. The footings of Building 7 are believed to be resting at least in part upon the glacial till which is overlain by the aforementioned cobbles and boulders and fill.

The level of subsurface water in the project working area is generally at river surface. To the south along the left bank terrace upstream of the dam and to the north along the right bank upstream of Station 10, the topography rises and groundwater levels can be expected to increase accordingly.

b. Availability of construction materials. Within the limits of the project several types of material will be removed. Required excavation for the dike exploration trench, channel widening and pier reconstruction are expected to produce silty, gravelly sands containing many cobbles and some boulders. Channel deepening is expected to produce materials varying from entirely cobbles and boulders from Stations 5 to 8 and 11 to 14, to a coarse mixture of sand, gravel, cobbles and boulders between Stations 8 and 11. The stone arch bridge when dismantled will produce a sizeable quantity of rough-cut blocks measuring 12 to 18 inches in thickness and weighing between 500 and 2,000 pounds.

Glacial till occurs as a thin and spotty mantle on the tops and upper slopes of the hills in this region. Limited amounts of silts are present in the lacustrine deposits but generally occur in thin layers or pockets or are overlain by relatively thick sections of silty sand and gravel. So-called "dirty" sands and gravels are available in large deposits throughout the area. One such deposit, although bouldery, is present near the bottom of the left slope of the valley immediately upstream of the project. Some quantity is also available in a knoll west of Station 10 on the right bank. However, large blocks, possibly bedrock, are exposed in this feature.

Gravel occurs in several deposits within 15 miles of the site, although the nearest commercial pit is in Ware, 5 miles away. The nearest known undeveloped deposits are just south of Warren on Route 19. It is expected that the local deposits are of limited quantity and gravel content. The existing commercial operations in Ware, Brookfield, Wilbraham and Ludlow produce gravel by crushing and/or washing and blending. Both pit-run material and concrete aggregates can be produced. Aggregates from a number of nearby sources have been tested and approved for use in concrete for civil works construction.

Quarry-run and crushed stone for aggregate are produced in Westfield and other quarries 35 miles or more from the site. Large piles of wasted quarry-run type rock occur adjacent to the Massachusetts Turnpike, just south of West Warren, and in an abandoned quarry in Monson some 10 miles from the project. Cut stone is produced in Uxbridge which is 50 miles from the site.

c. Foundations, embankments and slopes - general. Foundation conditions for the proposed structures were determined by the explorations described above, by visual examination at the site and by information supplied by West Warren Industries personnel. Samples from the borings and test pit were examined in the laboratory and mechanical analysis and Atterberg Limits tests were performed on typical samples. A record of explorations appears on Plate No. 4 and a summary of laboratory test data is shown on Plate No. S-1 included in Appendix A.

d. Dike. The dike is a low earth structure 390 feet long and is shown in plan and section on Plate No. 3.

The foundation is a heterogeneous mixture of sand, gravel, silt and non-earthen but substantially stable substances. From explorations FD-1 and FA-1 it may be seen that at least the top five feet is man-made fill ranging from loose silty gravelly sand to moderately compact sandy silts and contains minor amounts of brick, glass, cinders and metal. The fill may be as thick as 10 to 20 feet since prior to the establishment of the Industries, the bank was probably south of the existing bank, and since the samples from FD-1 appear to indicate fill down to Elev. 527 which is the general river bottom grade in that vicinity. The upper portion of the fill is retained along the river by a wall of uncemented cobbles and small boulders. Subsurface water is approximately at river level.

The embankment, which will be a maximum of 7 feet above existing ground, has been designed without computed analyses to provide a conservatively safe structure. The dike section has been based on utilizing fairly well-graded non-plastic gravelly silty sand from local deposits containing, based on the minus No. 4 sieve fraction, between 20 and 50% finer than the No. 200 sieve. The impervious fill material will be obtained from a Contractor's selected borrow source. Any earth material excavated for the foundation cut-off free of ashes and debris which meets the gradation specification may be used as impervious fill material. If such material is more pervious than the specified, it will be placed on the dike embankment on the land side of the centerline. The fill material will be compacted with small rubber-tired rollers or crawler tractors to at least 95% of Standard Proctor. Slopes of 1 on 2 are considered to be satisfactory for stability. Seepage through the embankment will be negligible but the variable nature of at least the top five feet of the foundation and the possible existence of abandoned pipes necessitates the provision of a six foot deep exploration trench. It is intended that the trench be excavated by back-hoe and be backfilled with the same material as to be specified for the embankment. Compaction of the trench fill will be provided by placing friable materials and rolling the surface with the tracks of a crawler tractor. No detrimental settlement of the foundation or embankment is anticipated. Slope protection on the riverside will be provided by a 12-inch layer of quarry-run type stone having an average size of 30 to 50 pounds. This size conforms to the minimum requirements in CWEB 52-15. A 12-inch layer of bank run gravel will be used as bedding. The gravel will be graded to prevent movement of bedding or impervious materials and will contain at least 50% retained on the No. 4 sieve and of the component passing the No. 4 sieve, less than 15% finer than No. 200 sieve. Both the rock and gravel will be supplied by the contractor.

The wrap-around section will be composed of compacted and uncompacted impervious fill on the riverside, and uncompacted gravel fill and compacted impervious fill on the landside, as shown on Sections 1-1 and 4-4, Plate No. 5. The wall extends into the top of the embankment a distance of 10 feet. This distance plus the distance provided by the 1 on 2 embankment slopes provides ample embedment for seepage control, considering that the maximum hydraulic head on the structure will be 7 feet. The zone of "uncompacted" impervious fill below Elev. 592 is contiguous with the cover over the heel of the new wall. The contractor will be required to place the uncompacted fills in layers of 12" or less, and it is considered that compactive effort from placing and hauling equipment will preclude the necessity for specifying hand compaction and will consolidate the fills sufficiently to prevent future detrimental settlement. A 1 on 4 slope has been provided from the end of the wall eastward to facilitate passage of vehicular equipment and compaction from Elev. 529 up. Erosion control at the wrap-around toe consists of rock fill over gravel bedding extending at least five feet into the river channel. Soft river sediments will be removed before placing the gravel bedding.

In order to protect the existing retaining wall and to prevent any new permanent loads being imposed upon it, the dike toe has been set back approximately two times the estimated free height of the wall above the solid river bottom. Also, the contractor will not be permitted to operate heavy equipment on the riverside of the dike upstream of the pumphouse.

e. "T" Wall. The "T" wall, as shown on Plate No. 5, is 120 feet long, about 25 feet high, and contains five substantially different monoliths.

The general soil conditions in the river section of the proposed wall, as estimated from boring FD-2 and by probing and observation, consist of about one to six feet of loose organic river-deposited sand and gravel overlying about five feet of compact alluvial-like gravelly sand, which in turn is underlain by compact gravelly silty sand glacial till. The foundation conditions for the section south of the existing retaining wall are believed to be approximate, as described for the dike in subparagraph d above. Affecting the design and construction of the wall are the following existing structures: the foundation wall of Building 7, which is estimated to extend to about Elev. 518; the dam and south abutment, details of which are not known; an underwater retaining wall reportedly aligned between the northeast corner of Building 7 and the pumphouse upstream of the new wall; and the gate and headrace structure.

The new wall has been designed in accordance with the Flood Wall Manual and Change 2 thereto. In consideration of the assumption of a crack in the soil over the heel, calculations, using the formula on page S-10, indicate that the wall will move less than 0.1-inch under repetitive loading. In accordance with paragraph S-12, 5 feet minimum of granular non-cohesive impervious cover has been provided. The material will be the same as specified for impervious fill for the dike. Under those conditions, it does not seem probable that a crack will occur, but the design has been based on full hydrostatic pressure to the bottom of the heel except as discussed below.

In order to obtain the minimum required earth cover over the heel, the wall will be founded at Elev. 518 or about 7 feet below what is considered to be the average grade of the river bottom.

The soil immediately below Elev. 518 is a compact alluvial silty gravelly sand containing in the order of 10% fines and/or compact gravelly silty m-f sand glacial till containing in the order of 35% fines. It is estimated that the sand layer is not greater than three feet in thickness below Elev. 518 and at the location of Boring FD-2 is one foot thick. The sand layer is underlain by the glacial till. The till is essentially infinite in depth and increases in compactness with depth. For the purposes of this design, it is considered that both layers are infinite in areal extent. Estimated permeabilities of the alluvial sand and the till are 10 and 0.1 x 10-4 cm/sec, respectively. Shear strength (S) have been taken as $\emptyset = 30^{\circ}$ and c=o for both materials. Settlement will be negligible under the proposed loads. Allowable bearing pressures are estimated to be at least 3000 psf.

Control of seepage is necessary to insure stability of the new wall and to prevent possible damage to Building 7 under the design loadings. A five-foot minimum impervious cover has been provided riverside of the wall, and combined with the natural river materials forms an impervious zone extending upstream. At the toe, a large pervious section will be constructed with a width of four feet at the contact with the natural foundation soils. The pervious section will be composed of gravel or gravelly sand containing, of the component passing the No. 4 Sieve, less than 10% finer than the No. 200 sieve. Perforated BCCM pipes, which will be installed in the pervious section at Elev. 529, will prevent the new construction from increasing the water surface level against the existing foundation wall of Building 7 and will maintain the water level downstream of the new wall to that in the new channel. The pipes will be surrounded by 6 inches of 3/4-inch concrete aggregate (filter gravel). Piping at the toe of the new wall will not occur. It is obvious, except behind the gate structure, considering the dimensions and existence of the pervious section and that the material will be more pervious than the foundation, that the factor of safety is well over the required 2.0. If the following very conservative assumptions are made for the section in the gate monolith:

(1) No upstream earth cover above the foundation elevation;

(2) The foundation soil consists of silty gravelly sand 3 feet thick overlying relatively impervious glacial till;

(3) The permeability of the upper 3 feet of foundation soil is the same as the material in the downstream pervious section;

the excess pore pressure, due to seepage at the toe of the base when the maximum possible differential head of 15.5 feet occurs, will be less than 3.0 feet of head, which will be counterbalanced by 11 feet of submerged gravel and rock paving. For this condition the factor of safety against piping is greater than the required 2.0. To insure that no movement of the gravel fill occurs at the surface due to variations in the material, the bottom of the channel for a distance of 15 feet will be paved with closely laid cut stone, which is available at the site and will be the same as used in the channel walls.

Analyses for stability against sliding were made for Section 2-2, Plate No. 5 which is typical for 40 feet of the wall and is the most critical section except for Monolith 3, which is the gate structure. Two loading conditions were considered: (a) landward sliding with flood water to the top of the wall, the gates closed and tail water to the channel invert; and (b) riverward sliding with gates open and normal water levels. Since the foundation may be glacial till or a thin sand layer over glacial till, both foundation conditions were considered. The thin sand layer case was found in a preliminary trial to be more critical and, therefore, has been used for all the analyses. The analyses were made assuming a crack in the soil over the heel. The results of an analysis, assuming no crack, indicated that the assumption relative to the crack would have no significant effect upon the results. The method of planes, assuming a sliding block and a passive wedge, was used in all analyses. The flow net and computations for each case are included in Appendix A of this report. Section 2-2 has factors of safety of 5.1 against landward sliding and 3.4 against riverward sliding. Monolith 3 was analyzed for landward sliding only and has a factor of safety of 2.5, considering that passive resistance due to the surcharge weight of the new channel walls and gravel fill over the passive wedge will be developed.

f. Interior walls. Inside Buildings 7 and 11, "L" buttress type walls and subsurface drains have been designed, as shown in plan on Plate No. 3 and in section on Plate No. 5.

The foundation materials in both buildings are fills at least four feet thick. In Building 7, the fill probably consists of variable, loose layers of gravelly sand, sandy organic silt, cinders and brick fragments and silty sandy gravel. The samples from boring FD-3 indicate the fill to be at least 9 feet thick at that location. A test pit at the north wall of Building 11 showed the fill to be four feet of gravelly sandy silt overlying what appeared to be alluvial gravelly sand, natural ground. Little variation is expected in the foundation conditions of Building 11. The foundation walls along the river are mortared stone and contain a number of drain pipes and openings below the elevation of the Standard Project Flood.

Although the exact nature of foundation soils for the new walls is not known, it is anticipated that suitable bearing will be obtained at the elevations shown. Provisions will be made to extend the footing, if very soft or otherwise unsuitable material is encountered. To decrease the possibility of damage due to seepage and uplift on the floor, a subsurface drainage system is provided and any observable openings in the exterior of the existing foundation walls will be mortared and flap gates will be installed on the drains. A perforated pipe will be installed and surrounded by 6 inches of 3/4-inch concrete aggregate (filter gravel). Six inches of concrete sand (filter sand) will be provided as a filter between the soil and the filter gravel. Subsurface drains will be piped to new manholes in the buildings. Portable pumps will pump and discharge seepage water into the river.

g. Stone protection. The channel slopes and bottom will be protected with riprap as is shown in plan on Plate No. 3 and in sections on Plate Nos. 3 and 5.

Riprap paving has been provided for erosion control and to maintain a smooth condition from the dam downstream to Station 12+50 and above the right abutment of the dam. Stone sizes were selected on the basis of velocity as shown in paragraph 39. One thousand pound minimum stone will be used on the side slopes and apron, and 500 pound minimum stone will be used on the bottom. Blasted or cut stone will be permitted. For the side slopes and apron, it will be specified that the stone will be roughly rectangular or cubical in shape; the maximum length will be less than three times its least dimension; the voids greater than three inches in minimum dimension shall be chinked; and that the finished layer will be 18 to 24 inches thick. Such material and construction is common in the area as they essentially conform to the Massachusetts State Specification for Slope Paving at Bridges. For the channel bottom the same shape requirements will apply, but no chinking will be required and the layer will be 12 inches minimum. A bedding of bank-run gravel, as specified in subparagraph d above, will be used except where the natural foundation meets the bedding requirements. It may be noted that during the highest flood stages, water will flow around the dam, down the railroad tracks, and over the slope downstream of Station 4+50. It can be expected that some damage to the slope protection will occur, since the entire area will not be paved. The rock slope protection has not been extended to the railroad tracks due to interference to normal track drainage as afforded by the existing ballast sections and excessive costs resulting from required railroad operation during construction. In addition stone protection in the upper reaches would be effective only during extreme flood stages.

Between the existing mortared cut stone abutment wall of the dam and the railroad tracks, from Stations 3 to 5, is a fill composed of, as observed from the surface, a mixture of gravel, wasted crushed stone ballast and boulders. Despite the apparent coarseness of the mass, erosion has taken place to the extent that several blocks in the downstream end of the wall have been washed out. In order to protect against further erosion during floods approaching the design flood, an apron of 1000-pound minimum stones will be provided. Some fill will be required behind the wall at about Station 4+50, and it is proposed to use material excavated from the channel. The existing foundation material will serve as bedding.

h. <u>Channel side slopes</u>. The side slopes have been selected as the flattest possible within the areal limitations imposed by channel width and existing facilities, and consistent with providing reasonably stable foundations. Plate Nos. 3 and 5 show the slopes in plan and section.

On the right bank between Stations 4+50 and 8+00, the top one to two feet of existing bank is composed of a mixture of sand, gravel, and cobbles dredged from the river and a few rock blocks. The materials at depths to which excavation will be carried are believed to be of the two main types. One, roughly above the level of the existing channel bottom, is rock blocks dumped into a large eroded area created by the 1938 flood. The second, below river bottom and in the slopes between about Stations 7 and 8, is expected to be the natural soil, gravelly silty sand. During periods of heavy precipitation and high water, it is expected that some seepage exits through the natural soil slopes. A 1 on 2 slope has been selected to insure stability considering the types of materials, the railroad, and the probability of significant quantities of water flowing down the slope during the highest floods.

The right bank between Stations 8+00 and 10+00, as indicated by the samples from boring FD-1 and test pit FT-1, contains soils varying from gravelly silty sand to silty sandy gravel. No subsurface water was encountered. However, the topography and geology to the North indicate that groundwater can be expected to occur in the slopes during periods of high water and heavy precipitation. Within this reach are the bridge piers which will be reconstructed and the most critical portion, hydraulically speaking, of the channel. The 1 on 1.5 slope was selected on the basis of space limitations. As shown on Section 5-5, Plate No. 5, the required excavation will remove a considerable quantity of material from the riverside of the railroad. In order to obtain a moderate increase in stability, considering that seepage will occur, a rock fill layer has been provided. The contractor will be required to excavate and place protection stone in 50 foot reaches in order to minimize exposure of the raw 1 on 1 slope. Gravel bedding will be provided in this entire reach.

From Stations 10+00 to 13+00 on the right bank, the soils vary from silty gravelly sand to gravelly sandy silt. Based on the topography and geology, it is believed that little or no seepage will exit on the face of the proposed slope. A 1 on 1.5 slope has been selected due to space limitations and the impracticability and undesirable hydraulic conditions of transitioning for short reaches. Considering the type of soil and that no seepage will occur, it is believed the factor of safety against shear failure of the slope will be greater than 1.0 but not the desirable value of 1.5. There is a possibility that some maintenance will be required as a result of local sloughing in this reach.

The entire left bank consists of gravel, sand, cobbles and boulders. A 1 on 1.5 slope is considered adequate for stability based on the height of slope, the type of materials and seepage conditions, except at Building 11 as discussed below.

The foundation for Building 11, which is a 5-story brick wall factory building located between Stations 8 and 9, has been analyzed for stability to determine the width of berm necessary to provide adequate foundation conditions. The soil profile is believed to consist of fill inside the foundation walls, as described in subparagraph f above; several feet of compact sandy gravel alluvial sand under the foundation; and compact glacial till below the alluvial layer. The elevation of the top of till is not known, however, the failure plane, as shown by the computations included in Appendix A, is considered to pass entirely through the alluvial layer. A shear strength of $\phi = 35^{\circ}$ and c=o tsf has been selected for the alluvial layer. Stability analyses were made by the circular arc method and by use of Terzaghi's bearing capacity formulae. The dimensions and elevation of the selected berm, as shown on Plate No. 5 and by the computations included in Appendix A, are the result of these analyses. The factors of safety obtained by the circular arc method for the final and construction conditions are 1.62 and 1.51 respectively. The analysis for the final condition, with the river at the critical Elevation of 520, and for the construction condition are shown on the computation pages included in Appendix A. To further insure stability during construction, provisions will be made for careful excavation operation and the immediate placement of the riprap paving.

A berm has also been provided to insure adequate foundation conditions for the existing bridge piers just downstream of Building 11. The nature of the loads on and the extent of the pier footings are not known. No analysis has been made, but it is considered that a 7-foot berm at an elevation to which the footing is known to extend and a 1 on 1.5 slope will provide satisfactory foundation stability.

i. Bridge pier. In general, the existing soils in the vicinity of the bridge pier are gravelly sands with cobbles and boulders. Groundwater may be encountered near the bottom of the excavation.

Allowable bearing pressures will be in the order of 3000 psf, and no difficulty with settlement or sliding is anticipated. However, further explorations are considered necessary to determine the conditions at depths to be affected by the construction of the pier and underpinning of the existing bridge and provisions will be made for a drive-sample boring during the plans and specifications phase. Any necessary foundation stability analyses will be made at that time.

41. STRUCTURAL DESIGN

a. Purpose & scope. This section of the design memorandum presents the design criteria and basic data and assumptions used in the structural design of Tee and "L" shaped buttress walls and gate section. A brief description of the loading conditions and assumptions used is included to show the design procedure. Typical computations are included showing the maximum conditions for the critical walls. Additional computations following the same procedure will be made wherever warranted by a change in loading or a reduction in section.

b. Design criteria.

(1) General. All allowable working stresses conform to those specified in the Engineering Manual EM 1110-1-2101, "Working Stresses for Structural Design", dated 6 January 1958. Loading conditions, design assumptions and other design criteria are based on the following applicable parts in the Enginnering Manual for Civil Works issued by the Chief of Engineers: Standard Practice for Concrete, Part CXX, October 1953 and Wall Design, Flood Walls Part CXXV, Chapter 1, January 1948. Accepted engineering practice has been employed in cases where the Engineering Manual for Civil Works does not apply.

(2) Concrete. The following table lists the concrete and reinforced concrete stresses used in the design of these walls. In each case, the Civil Works Manual exposure classification A (applicable to structures subject to moderately severe weather exposure) has been used.

Flexure	Lbs. per Sq. In.
Extreme fiber stresses in compression	1,050
Extreme fiber stresses in tension (Plain concrete)	60

	Lbs. per Sq. In.
Shear - (v)	90
Bond - (u) Deformed bars	
Top bars	210
All others	300
Modular Ratio - (n)	10

(3) Reinforcement

(a) Grade and Working Stresses. All reinforcement in the walls, including temperature and shrinkage reinforcement, is designed for the working stresses of new billet steel, intermediate grade, deformed bars which is 20,000 psi in flexural tension. The reinforcement shall conform to the requirements of Federal Specification QQ-S-632, Type II, Grade C and to ASTM A-305-56T.

(b) Minimum Cover for Main Reinforcement

Bottom of Footing	4 inches
All other	3 inches

The concrete covering, spacer rods and similar secondary reinforcement may be reduced by the diameter of such rods.

(c) Spacing. The clear distance between parallel bars will not be less than $1\frac{1}{2}$ times the diameter of round bars except that in no case will the clear distance between parallel bars be less than 1 inch, or $1\frac{1}{2}$ times the maximum size of the coarse aggregate.

(d) Splices. All splices will be lapped 30 diameters to develop by bond, the total working strength of the bars. Splices in the main reinforcement at points of maximum moment have been avoided in the design.

(e) Temperature and Shrinkage Reinforcement. Temperature and shrinkage reinforcement will be provided in slabs or walls where the main reinforcement extends in only one direction. Such reinforcement will provide for a ratio of steel area to concrete area (bd) of 0.002 with a minimum of .001 in each face up to a maximum of #6 bars at 12" c. c.

(4) Increase in Normal Working Stresses. There will be no conditions under which working stresses will be increased.

(5) Waterstops. Rubber or polyvinyl waterstops will be used at all wall construction joints. Waterstops will generally be of the 2 inch center dumbell type in the wall stem and the "U" type in the base slab.

c. Basic data and assumptions

(1) Dead Loads. The following unit weights for material have been used:

Material	Unit Weight (lbs/Cu. Ft.) Dry	Saturated	Moist	Submerged	Ko
Fill	115	135	130	72.5	406
* Fill	-	125	120	62.5	-
Granite	145				
Concrete (Plain and Reinforced)	150				
Brick Masonry	120				

*These values were obtained subsequent to preparation of all computations except the Gate Section and have been used only for the Gate Section.

(2) External Water Pressure. In cases where hydrostatic pressure affects the design of a structure, it has been assumed to act over the entire area in question under the full head available.

(3) Earth Pressure. Earth pressure used against the structure has been determined in general in accordance with Part X, Structural Design, Chapter 9, Retaining Walls.

(4) Earthquake Forces. Earthquake forces are considered negligible and are not included in the design.

(5) Ice Pressure. Horizontal forces due to the expansion of ice have been disregarded.

(6) Wind Pressure. Wind pressure on the wall is negligible and has been disregarded in the design.

(7) Location of Resultant. In the investigations for stability of the walls, the resultant of the horizontal and vertical forces has been held within the middle half when flood is at the top of the wall and within the middle third when the flood is within three feet of the top of the wall.

(8) Factor of Safety Sliding. A factor of safety against sliding of at least 1.5 has been obtained.

(9) Bearing Pressure. The limiting bearing pressure is 3,000 lbs. per sq. ft.

d. Wall investigations

(1) Description

(a) Tee Wall. There is approximately 120 feet of Tee Type Cantilever Wall extending from the end of the dike to Building 7. This includes a center section with two 4' x 5' sluice gates which is described in the next paragraph. The maximum Tee Wall Section has a stem 26'-6" from bottom of base to top of wall and a base width of 20'-0". A minimum of earth cover of 7'-0" has been obtained on the riverside and 17'-6" on the landside. The founding of the bottom of the base at this depth was necessary in order to reach firm bearing. Because of the earth cover on the landside and the depth of embedment, a key was found unnecessary.

(b) Gate Section. The gate section monolith is 25'-0" long and contains two 4' x 5' sluice gates. The height of wall, and base width match the adjacent Tee Walls. The amount of earth cover over the base is 7'-0" on the riverside and 9'-0" on the landside for the 12-foot width of the channel. Buttressess are required on both sides of the wall stem in order to form the landside channel and to provide for a stop log closure on the riverside.

(c) "L" Buttress Type Wall Within Building 7. There will be approximately 350' of an "L" buttress type wall within the basement of Building 7. This wall will be cast against the building foundation and has been computed for stability discounting any help from the building foundation. This wall is 6' high and has an effective base width of 5'.

(d) "L" Buttress Type Wall Within Building 11. The north end of Building 11 requires approximately 70' of "L" buttress type wall similar to that in Building 7. The design of this section is not included herein but will be similar to that in Building 7.

(e) New Concrete Bridge Pier Footing. The existing stone bridge pier footing on the north side of the channel will be removed and replaced by a new concrete footing. This will be accomplished by providing temporary supports for the existing steel bridge supports during the construction operation. The new footing will be mass concrete and no special structural design consideration will occur.

(2) Loading Conditions

(a) Loading I - Channel full, water surface to top of wall. Path of creep for uplift considerations, considered starting at the bottom of the base slab and ending at the ground surface on the landside. Resultant to fall within the middle half of the base.

(b) Loading II - Same as I except path of creep ending at the channel bottom on the landside.

(c) Loading III - Same as I except water surface 3 feet below the top of the wall and the resultant to fall within the middle third of the base.

(d) Loading IV - Same as I except water surface 3 feet below top, path of creep ending at the channel bottom and the resultant to fall within the middle third of the base.

(e) Loading V - Channel empty to gate sill and soil at the back of the wall in a moist condition. Uplift considered uniform across the base and the resultant to fall within the middle third.

(3) Investigation results.

(a) Tee wall. The maximum bearing pressure was found to be 2,919 lbs. per square foot under Case II. The base width selected was such that for Cases I and II the resultant falls just within the middle half and for Cases III and IV at about the third point. Case IV was just outside the third point but was considered close enough to proceed with the selected width. The steel reinforcement for the stem and base slab was determined in the conventional manner.

(b) Gate section. The maximum bearing pressure was found to be 2,160 lbs per square foot under Case I. The base width is the same as for the Tee Wall. The resultant under Case I loading falls within the middle half and will be within the middle third of the base for Case III loading. The design of the wall and base slab will be accomplished as slabs supported by the buttresses. The effect of the slab reactions on the buttresses will be investigated to determine the buttress steel.

(c) "L" Buttress type wall. Because of the low height and the shape of this wall, it was investigated for Case I loading only. It was found that the bearing pressure was only 725 lbs. per square foot and that the resultant falls just outside the middle third. Steel reinforcement required is only nominal.

42. RELOCATION OF UTILITIES

The plan of improvement will require the relocation of overhead power lines and six electric poles owned by the Worcester County Electric Company. Four poles on the left bank upstream of the dam are to be relocated behind the new dike. One pole on the stone arch bridge is to be relocated to the right bank behind the bridge abutment, and one pole at the northwest corner of Building 11 will be relocated to the South Street bridge. There are no changes contemplated for existing sewer, water, or drainage lines except for the construction of flap valves at culvert and pipe outlets in the north wall of Building 7.

R. MULTIPLE-PURPOSE FEATURES

The West Warren Local Protection Project is designed solely for flood protection and contains no multiple-purpose features.

S. RECREATIONAL DEVELOPMENT

The plan of improvement is solely for flood protection and contains no recreational features or future developments for recreation.

T. ESTIMATES OF FIRST COSTS AND ANNUAL CHARGES

43. GENERAL

Estimates of Federal and non-Federal first costs and annual charges are given in Table 7. These estimates have been prepared on the basis that local interests would bear the cost of the following: relocations and alterations to utilities, excluding gates and valves required to seal off existing drain lines; removal and replacement of company-owned bridges; new intake gates and channel walls; furnish all lands, water rights and rights-of-way necessary for project construction including disposal areas; and operate and maintain the project after completion. Unit prices used in estimating costs are based on average bid prices for similar work in the same general area. The prices are based on 1961 price levels and include minor items of work which are not separately detailed in the cost estimates.

44. BASIS OF COST ESTIMATES

Detailed cost estimates have been made upon the basis of a design which would provide an economical and safe structure for the particular site. Estimates of quantities have been made upon the basis of neat outlines of the proposed designs and foundation requirements. Both financial and economic costs were computed as outlined in Corps of Engineers Engineering Manual 1120-2-104.

45. CONTINGENCIES, ENGINEERING, SUPERVISION AND ADMINISTRATION

To cover contingencies, estimates of construction costs have been increased by 15 percent. The cost of future engineering and design has been taken as 8.6 percent of the construction costs. The cost of supervision and administration has been taken as 7.9 percent of the combined construction and engineering costs.

TABLE 7

ESTIMATES OF FIRST COSTS AND ANNUAL CHARGES

LOCAL PROTECTION, WEST WARREN, MASS.

FIRST COST
(1961 Base)

FEDERAL

Item	Quantity	Unit	Unit Cost	Amount
Site preparation	1	Job	L. S.	$ 1,000
Stream control	1	Job	L. S.	20,000
Excavation-unclassified	15,100	c. y.	$1.70	25,670
Rock excavation	1,000	"	5.00	5,000
Impervious earth fill	2,400	"	2.00	4,800
Gravel bedding	3,700	"	3.00	11,100
Cover stone (1,000 lb)	2,400	"	11.00	26,400
Stone paving (500 lb)	1,700	"	10.00	17,000
Rockfill	1,100	"	6.00	6,600
Gravel fill	1,300	"	3.00	3,900
Concrete walls	650	"	52.00	33,800
Concrete pier footing	370	"	30.00	11,100
Steel sheet piling	2,600	s. f.	5.00	13,000
Underpin & support bridge	1	Job	L. S.	14,000
Anchor bolts, plates & misc.	1	"	"	500
Topsoiling & seeding	1	"	"	500
Pipe extensions & flap valves	5	Ea.	200.00	1,000
Remove and repair conc. floors	550	s. y.	4.70	2,585
6" Perforated toe drain	400	l. f.	3.70	1,480
8" Perforated pipe	100	l. f.	2.20	220
Portable pumps and hose	2	ea.	600.00	1,200
Misc. drainage facilities	1	Job	L. S.	900
	Total			$201,755
	Contingencies, 15%			30,245
	Total Construction Cost			$232,000
	Engineering & Design			46,080 (1)
				$278,080
	Supervision & Admin.			21,920
Total Estimated Federal First Cost				$300,000

(1) Includes $26,080 for prior and present reports.

TABLE 7 (Cont.)

NON-FEDERAL

Item	Amount
New gates and channel	$ 22,000
Remove and replace oil pipeline bridge	10,000
Lands	5,000
Remove stone arch bridge	5,000
Relocate power line and poles	2,500
Remove steam line and support	500
Total Estimated Non-Federal First Cost	$ 45,000
Total Estimated Project First Cost	$345,000

ANNUAL CHARGES

FEDERAL

Interest (2.625% x $300,000)	$ 7,900	
Amortization (.989% x $300,000)	3,000	
Total Federal Annual Charges		$10,900

NON-FEDERAL

Interest (4% x $45,000)	$ 1,800	
Amortization (.655% x $45,000)	300	
Maintenance	400	
Total Non-Federal Annual Charges		2,500
Total Annual Charges		$13,400

$$\text{Benefit-Cost Ratio} = \frac{\$17,400}{13,400} = 1.3 \text{ to } 1.0$$

46. BASIS OF ANNUAL CHARGES

The estimates of financial and economic annual charges were based on the use of public funds for the total investment over a period of 50 years. Federal annual charges include 2.625 percent of the total investment for interest and 0.989 percent for amortization. Non-Federal annual charges include 4.0 percent of the construction investment with 0.655 percent for amortization plus maintenance costs. The non-Federal interest rate was determined after an investigation had been made of the borrowing power and repayment ability of cities and towns in the Chicopee River basin. Maintenance charges were based on the particular site conditions and previous experience with similar projects.

U. ESTIMATES OF BENEFITS

Average annual flood damage prevention benefits, taken as the difference between average annual losses under existing conditions and those losses remaining after construction, amount to $17,400. It is not anticipated that the project will result in any increased utilization of enhancement benefits. Details of the derivations of benefits are set forth in Appendix C.

V. COMPARISON OF BENEFITS AND COSTS

Average annual benefits for the West Warren Local Protection Project are estimated at $17,400 and average annual costs are estimated at $13,400. The resulting ratio of benefits to cost is 1.3 to 1.0.

W. PROJECT FORMULATION AND ECONOMIC JUSTIFICATION

The Division Engineer finds that severe floods have caused extensive damages to the lands and structures along the Quaboag River in West Warren. He concludes that a plan of improvement consisting of dikes, flood walls, channel excavation, removal of existing bridge restrictions, modification to one bridge pier, and stone slope protection would adequately protect this area against future flooding. Project formulation resolved itself into only one plan of protection which had not only economic justification but also afforded construction feasibility and compatibility with buildings in the area. Protection can be provided most suitably by the plan as submitted herein for approval.

The major feature considered in the formulation of various alternate plans of protection concerned improvements in the vicinity of Station 9+0. This critical area is bounded on the left by the corner of the five-story Building 11 and on the right by the north piers of the overhead South Street bridge. In order to make effective improvements upstream of the dam, and to sufficiently reduce tailwater conditions downstream of the dam so as to relieve hydraulic pressures from inducing failures to the north walls of Buildings 7 and 11, it was necessary to increase the channel area at this location. Channel deepening and widening along the right bank of the river was found to sufficiently reduce flood profiles to a level whereby interior buttress walls could sustain hydraulic pressures. The construction of higher flood walls on the outside of Buildings 7 and 11 resulted in an uneconomical alternate plan, owing to much greater volume of concrete and delicate construction procedures involved in the shoring of existing buildings.

Channel excavation along the right bank does, however, necessitate the removal and replacement of the stone rubble piers of the South Street bridge. Detouring of highway traffic and temporary underpinning of the bridge deck will be required to successfully install a new concrete pier as shown on Plate No. 5. Total project costs of the recommended plan are estimated at $345,000, of which $300,000 represents the Federal share and $45,000 the non-Federal share. The plan of protection will yield average annual benefits of $17,400 as against annual costs of $13,400, producing a benefit-to-cost ratio of 1.3 to 1.0.

X. SCHEDULES FOR DESIGN AND CONSTRUCTION

47. DESIGN

It is estimated that preparation of contract plans and specifications for the project will require 6 months. The estimated cost is $20,000.

48. CONSTRUCTION

Construction of the project can be accomplished under a single contract to be completed in a 10-month period. All funds for the construction of the project will be requested upon evaluation of bids received. Federal expenditures are estimated as follows:

Allotments to date (Reconnaissance Report and Detailed Project Report)	$ 26,080
Plans and Specifications	20,000
Construction, Engineering during Construction, Supervision and Administration	253,920
Total Estimated Federal First Cost	$300,000

Y. OPERATION AND MAINTENANCE

Maintenance of this project will be the responsibility of the local interests. Periodic inspections will be made to assure that adequate maintenance is performed in accordance with regulations prescribed by the Secretary of the Army. It is estimated that maintenance of the project will cost local interests $400 annually. An operation and maintenance manual will be provided the Town of Warren upon completion of the project.

Z. LOCAL COOPERATION

In accordance with Public Law 685, 84th Congress, adopted 11 July 1956, local interests would be required to provide without cost to the United States all lands, easements and rights-of-way necessary for the construction and operation of the project; hold and save the United States free from damages due to construction work; obtain from the New York Central Railroad Company an agreement to release the Town from any claims for damages due to construction; and maintain and operate all the works after completion in accordance with regulations prescribed by the Secretary of the Army. The responsibility for furnishing disposal areas for excavated materials not used in the project and for the modification and removal of bridges and relocation of utilities would rest with local interests under the requirements of lands, easements, and rights-of-way. Local interests would also be required to furnish the added assurance that they would contribute to the United States all necessary funds over and above the Federal cost limitation of $400,000. State and Town officials have indicated a willingness to fulfill conditions of local cooperation. Letters from State and Town authorities which constitute preliminary assurances are included in Appendix C of this report.

AA. COORDINATION WITH OTHER AGENCIES

Plans for the local protective works in West Warren have been reviewed by officials of the Town of West Warren, West Warren Industries, and the Commonwealth of Massachusetts. Their endorsement of the proposed plan is indicated by letters in Appendix C. Copies of comments from State and Federal agencies are included as exhibits in Appendix C of this report. The project has no effect on hydroelectric power generation, recreation, pollution abatement, fish migration or other collateral water resource uses.

BB. CONCLUSIONS

Investigations and studies for the local protection project covered by this report lead to the following conclusions:

a. West Warren Industries faces the threat of heavy damages in future floods. A recurrence of the flood of record would cause damages of $1.1 million.

b. The desires of local interests are for the best form of protection that can be afforded to their principal industries which would thereby secure the economic base of the Town.

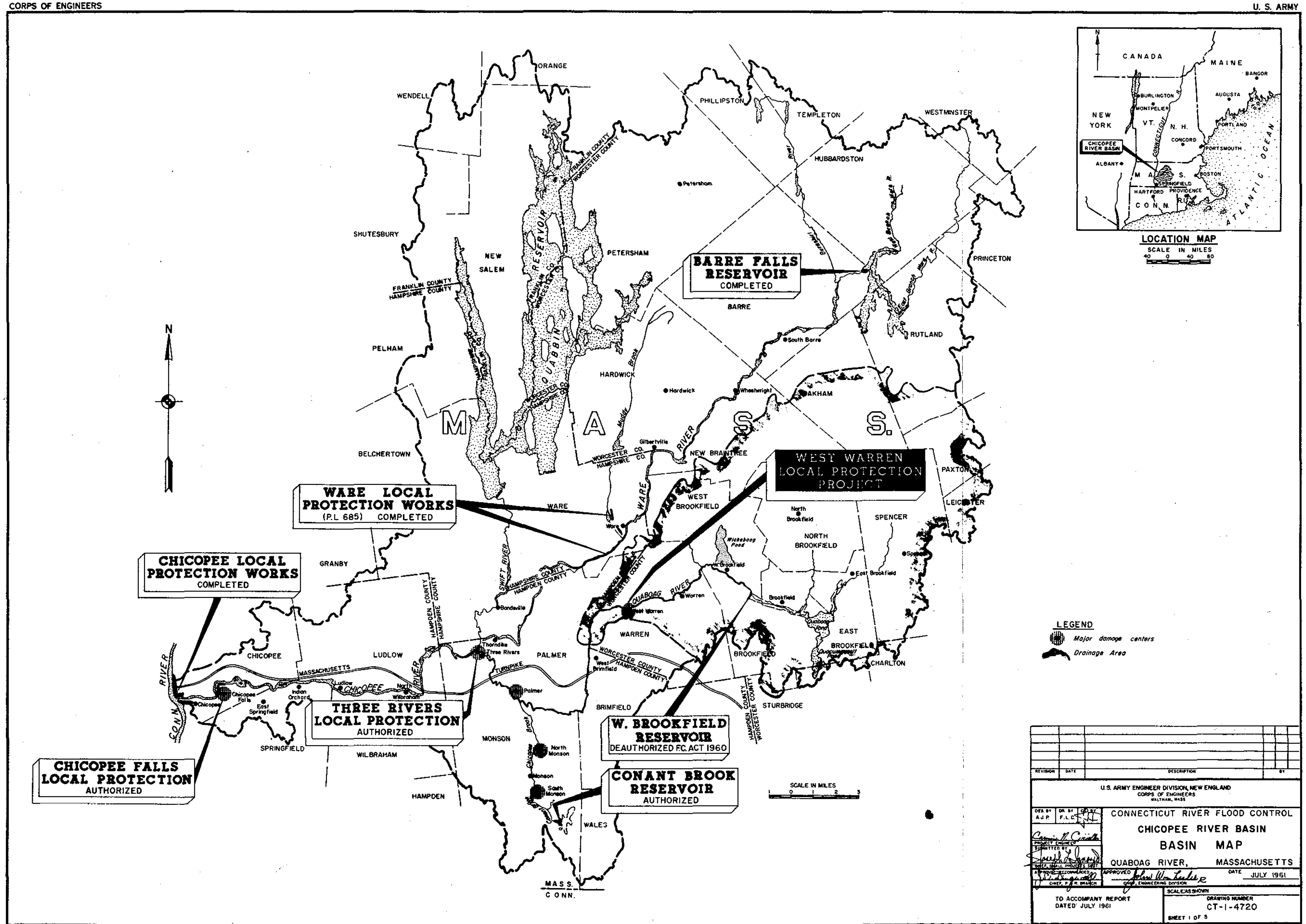
c. Protection can be provided most suitably by the proposed plan at a total estimated first cost of $345,000.

d. The project is economically justified by the ratio of annual benefits to annual costs of 1.3 to 1.0.

e. The threat of recurrent damaging floods makes it desirable to construct the project as soon as possible.

CC. RECOMMENDATIONS

It is recommended that the project, as submitted in this report, be authorized by the Chief of Engineers under the provisions of the Flood Control Act of 1948, as amended, and that additional funds be allotted in the amount of $20,000 for preparation of plans and specifications. Funds for construction will be requested upon receipt and analysis of bids for construction.



LOCATION MAP
SCALE IN MILES
CANADA
MAINE
NEW YORK
VT.
N. H.
MASS.
CONN.
ATLANTIC OCEAN
CHICOPEE RIVER BASIN
BARRE FALLS RESERVOIR
COMPLETED
WEST WARREN LOCAL PROTECTION PROJECT
WARE LOCAL PROTECTION WORKS
(P.L 685) COMPLETED
CHICOPEE LOCAL PROTECTION WORKS
COMPLETED
THREE RIVERS LOCAL PROTECTION
AUTHORIZED
CHICOPEE FALLS LOCAL PROTECTION
AUTHORIZED
W. BROOKFIELD RESERVOIR
DEAUTHORIZED F.C. ACT 1960
CONANT BROOK RESERVOIR
AUTHORIZED
QUABBIN RESERVOIR
QUABOAG RIVER
WARE RIVER
SWIFT RIVER
CHICOPEE RIVER
CONN. RIVER
MASSACHUSETTS TURNPIKE
LEGEND
Major damage centers
Drainage Area
SCALE IN MILES
REVISION DATE DESCRIPTION BY
U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.
CONNECTICUT RIVER FLOOD CONTROL
CHICOPEE RIVER BASIN
BASIN MAP
QUABOAG RIVER, MASSACHUSETTS
DATE JULY 1961
TO ACCOMPANY REPORT DATED JULY 1961
DRAWING NUMBER CT-1-4720
SHEET 1 OF 5

PLATE NO. 1

CORPS OF ENGINEERS

U. S. ARMY

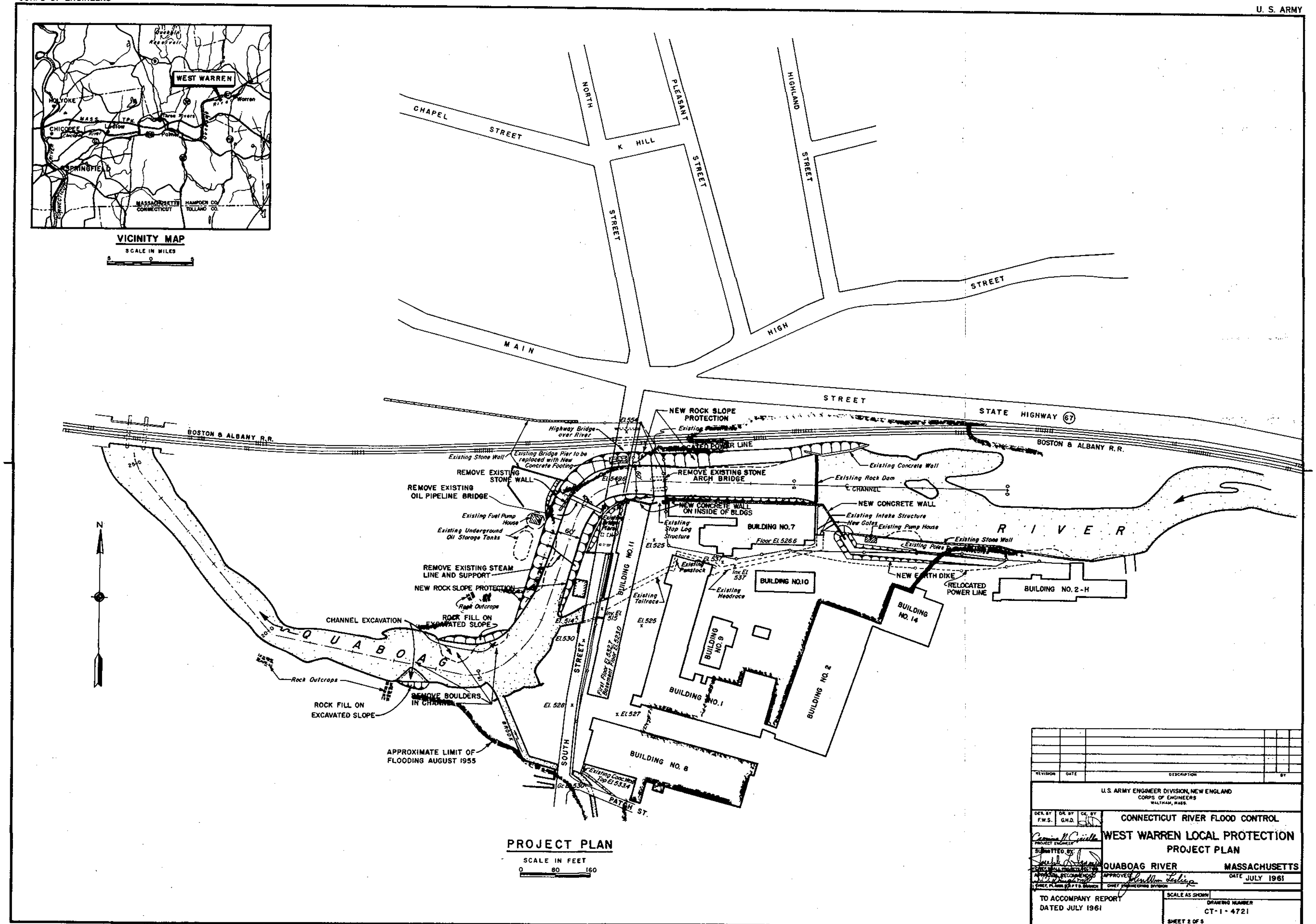


PLATE NO. 2

CORPS OF ENGINEERS

U. S. ARMY

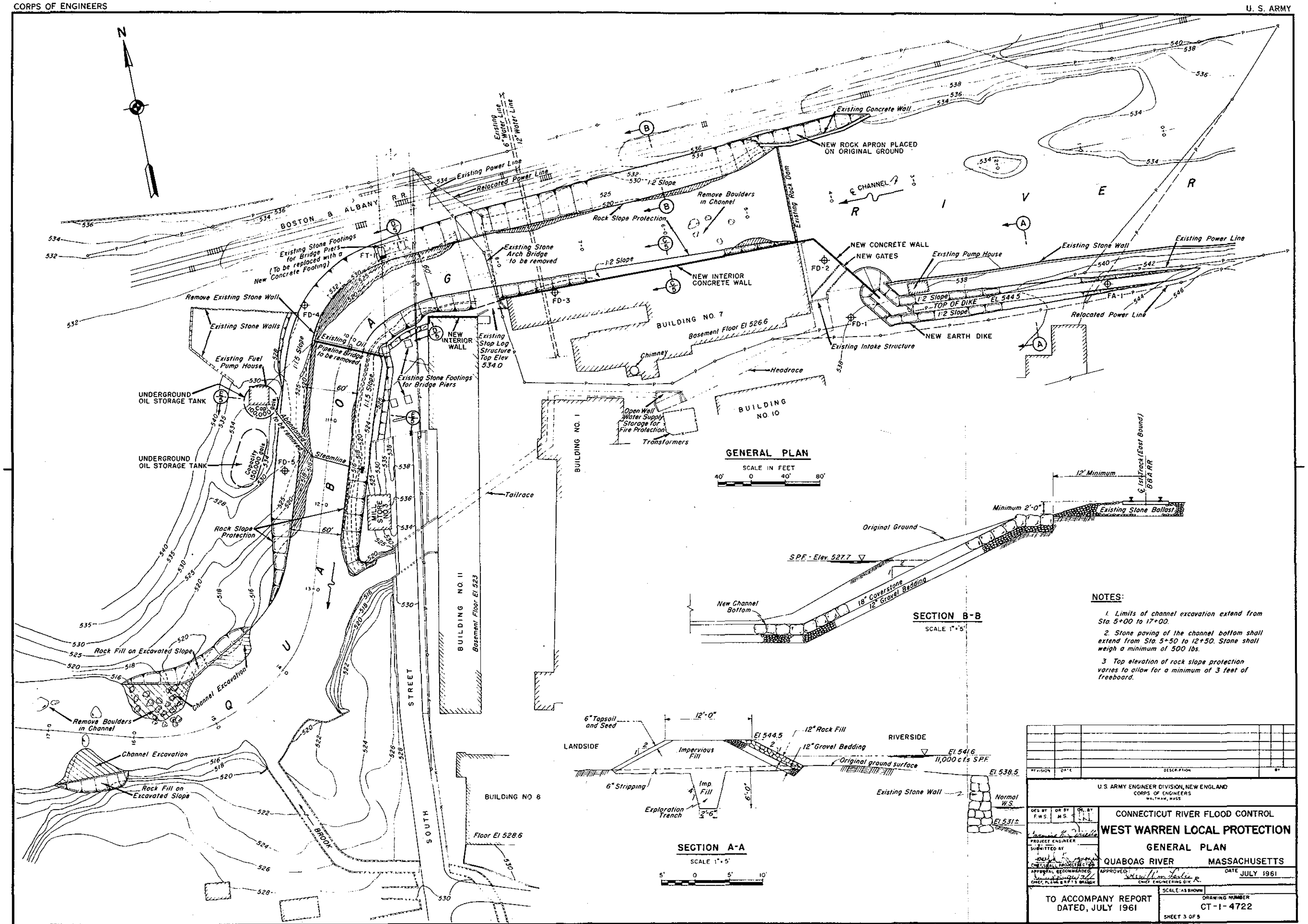
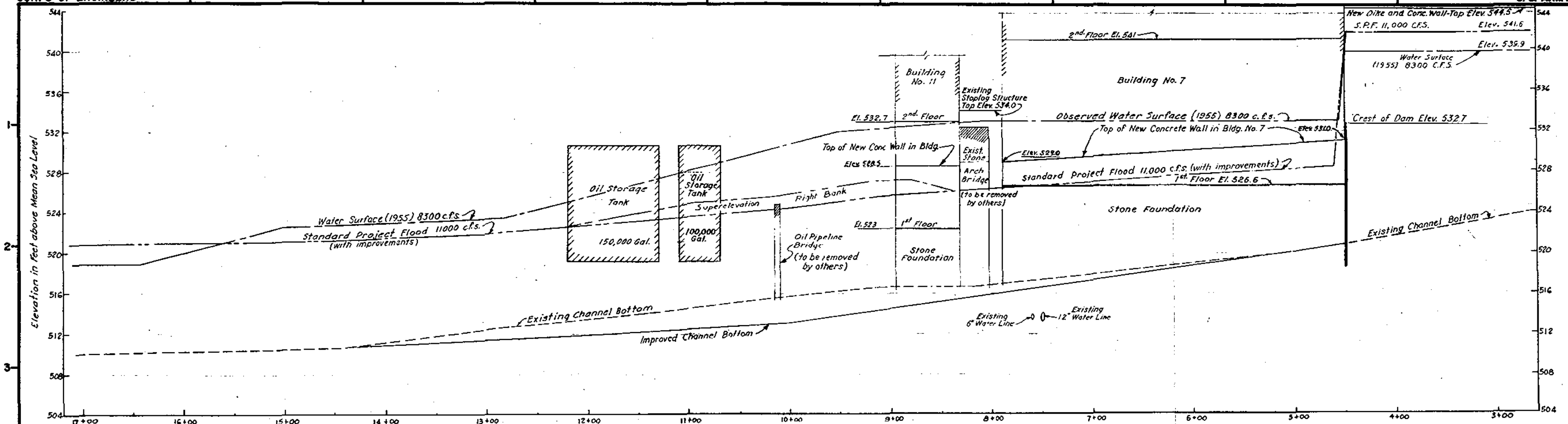


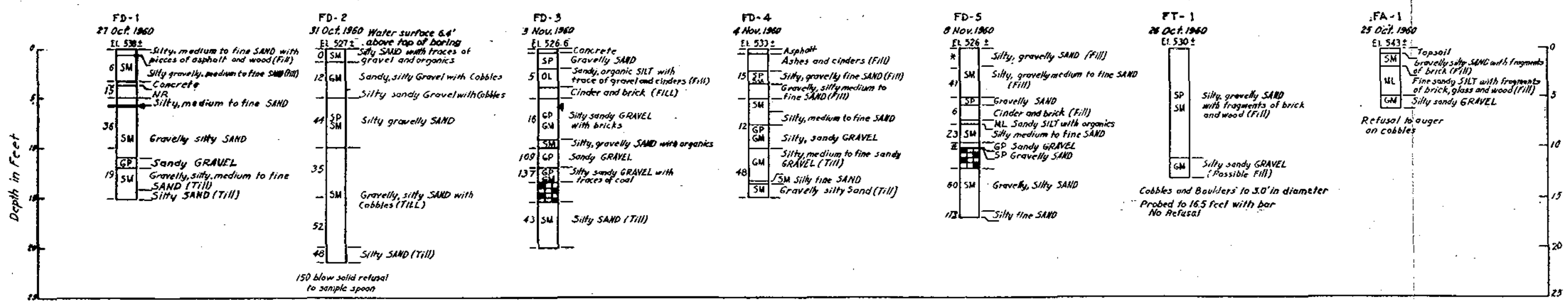
PLATE NO. 3

CORPS OF ENGINEERS — U. S. ARMY



℄ PROFILE

SCALE: HORIZONTAL 1" = 40'-0"
VERTICAL 1" = 4'-0"



NOTES

Elevations refer to Mean Sea Level Datum.
For location of Subsurface Explorations see Plate No. 3.

RECORD OF EXPLORATIONS

SCALE: VERTICAL 1" = 5'

LEGEND FOR GRAPHIC LOGS

FD-5	Foundation Test Boring
FA-1	Foundation Hand Auger Boring
FT-1	Foundation Test Pit
3 Nov 1960	Date Exploration completed
EL 526	Elevation of ground surface at time of exploration
	Subsurface water level in boring at time of exploration
SM	Group letter symbol according to Unified Soil Classification
NR	No recovery or unsatisfactory soils samples recovered
25	Blows per foot of penetration considered most representative for each sample drive using a 350 pound hammer with a free fall of about 18" on a 1⅜" I.D. or 2" O.D., 2" I.D. or 2½" O.D. and or 2⅞" I.D. or 3" O.D. size sample spoon equipped with a beveled and sharpened drive shoe
X	Blow counts not recorded or not considered representative
	Cobble or boulder (Core-drilled)
	Cobbles or boulders, continuous or nested (Core drilled) and/or blasted and chopped

REVISION	DATE	DESCRIPTION

U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

CONNECTICUT RIVER FLOOD CONTROL

WEST WARREN LOCAL PROTECTION

PROFILE AND EXPLORATIONS

QUABOAG RIVER — MASSACHUSETTS

DES. BY E.P.S. DR. BY M.S. CK. BY

APPROVED — DATE JULY 1981

SCALE: AS SHOWN

TO ACCOMPANY REPORT DATED, JULY 1961

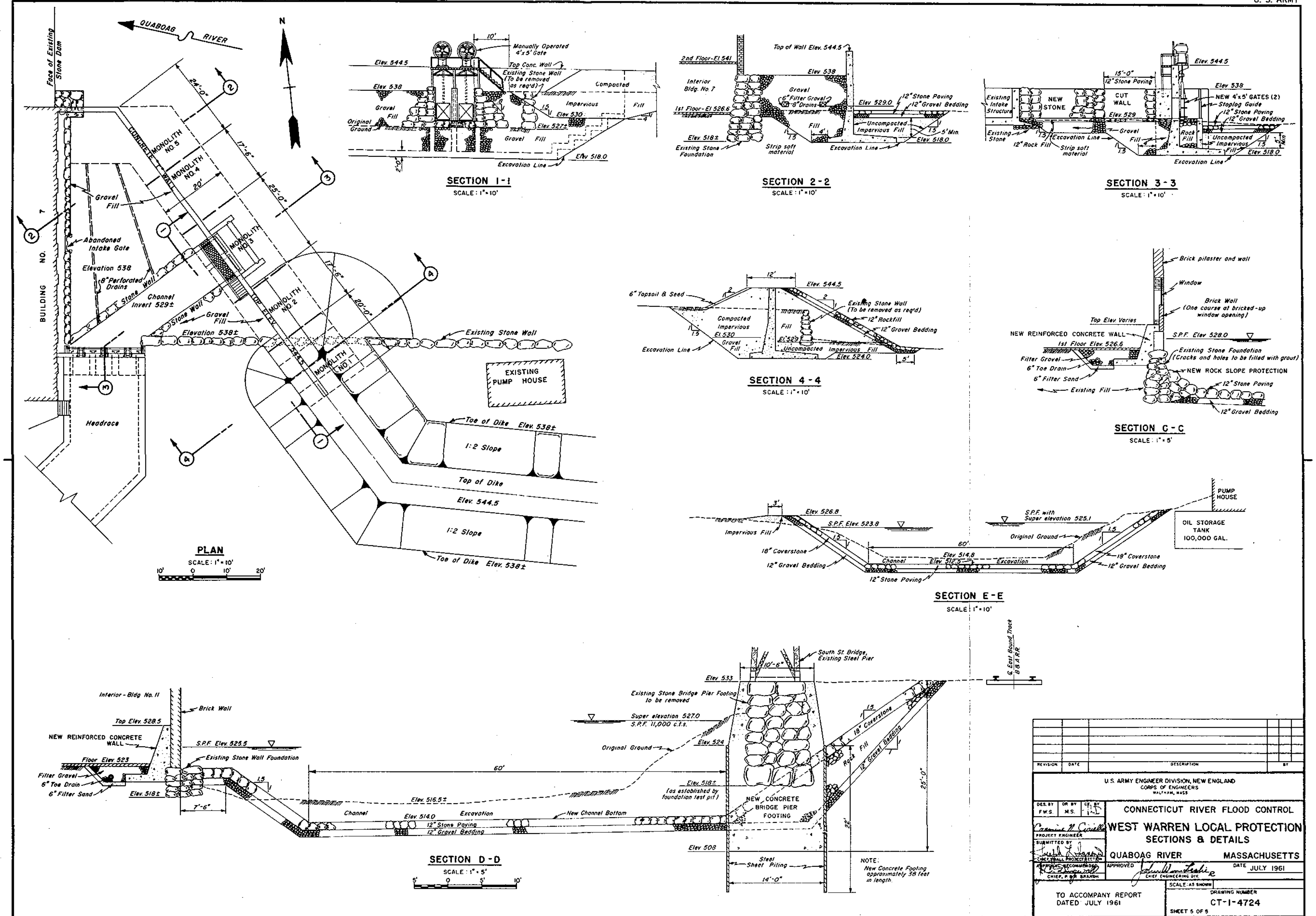
DRAWING NUMBER CT-1-4723

SHEET 4 OF 5

PLATE NO. 4

CORPS OF ENGINEERS

U. S. ARMY



REVISION	DATE	DESCRIPTION	BY

U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

CONNECTICUT RIVER FLOOD CONTROL
WEST WARREN LOCAL PROTECTION
SECTIONS & DETAILS
QUABOAG RIVER — MASSACHUSETTS

DES. BY F.W.S. — DR. BY M.S.

PROJECT ENGINEER — SUBMITTED BY — APPROVAL RECOMMENDED — CHIEF, P. & R. BRANCH — APPROVED — CHIEF, ENGINEERING DIV.

DATE JULY 1961

SCALE: AS SHOWN

DRAWING NUMBER CT-1-4724

SHEET 5 OF 5

TO ACCOMPANY REPORT DATED JULY 1961

PLATE NO. 5

APPENDIX A

DESIGN COMPUTATIONS

INDEX TO SHEETS

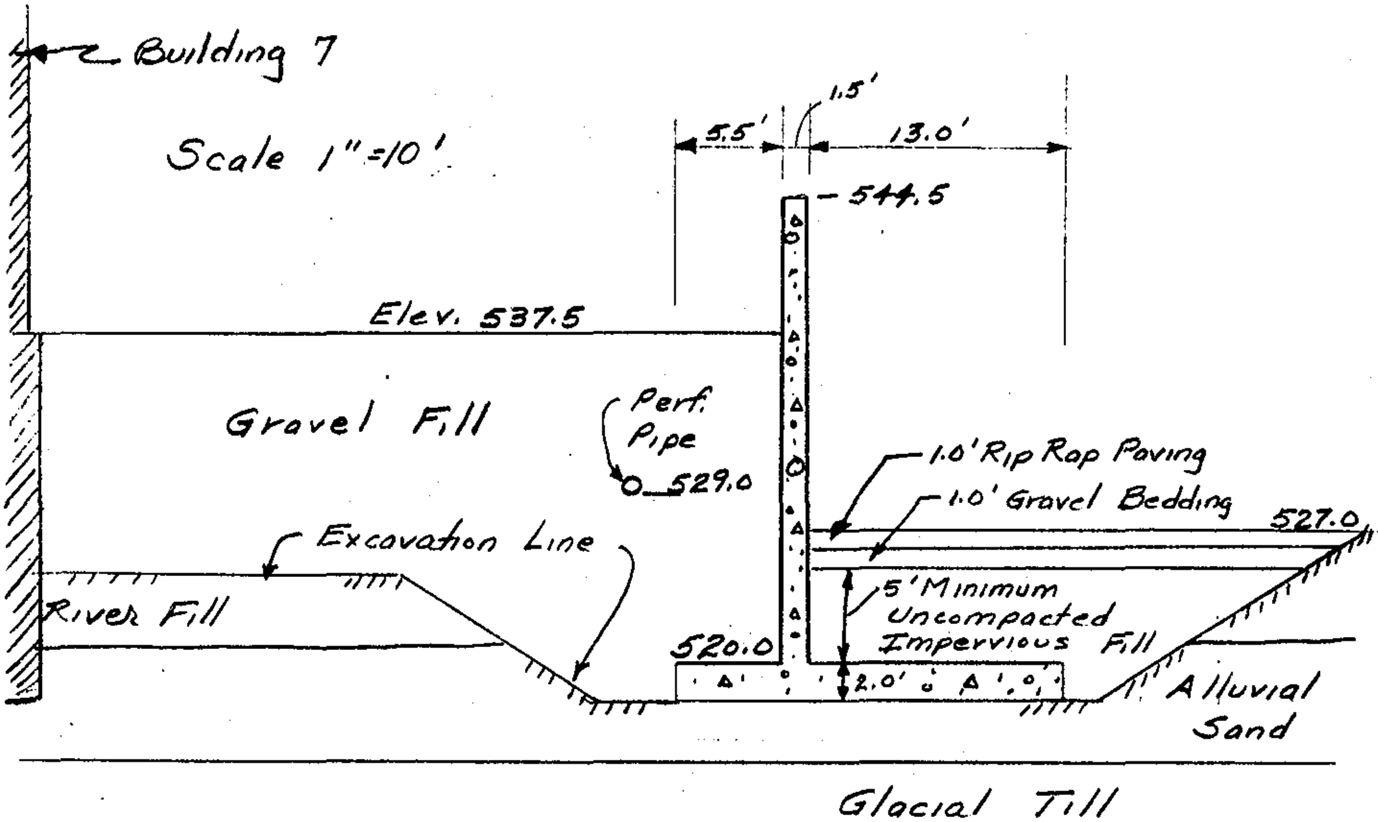
INDEX TO SHEETS

PLATE

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT W. WARREN, MASS. LOCAL PROTECTION - "T" WALL
COMPUTATION Foundation Stability Analysis - MONOLITH 2
COMPUTED BY ATL CHECKED BY ATL DATE 8 June 61



MATERIAL DATA					
Material	Unit Wt. State	Unit Wt. pcf	Shear Strength Ø - Deg.	Shear Strength C - TSF	Permeability k - x10⁻⁴ cm/sec
Glacial Till	-	-	-	-	0.1
Alluvial Sand	-	-	30	0	10
Gravel Fill	Moist	130	30	0	10
" "	Sat.	135			
" "	Sub.	72.5			
Uncomp. Imp. Fill	Sat.	125	25	0	1
" " "	Sub.	62.5			
Gravel Bedding	Sat	125	25	0	-
" "	Sub	62.5			
Rip Rap Paving	Sat	125	25	0	-
" " "	Sub	62.5			
Concrete	Sat	150	-	-	-
"	Sub	82.5			

LOADING CONDITIONS ANALYZED:

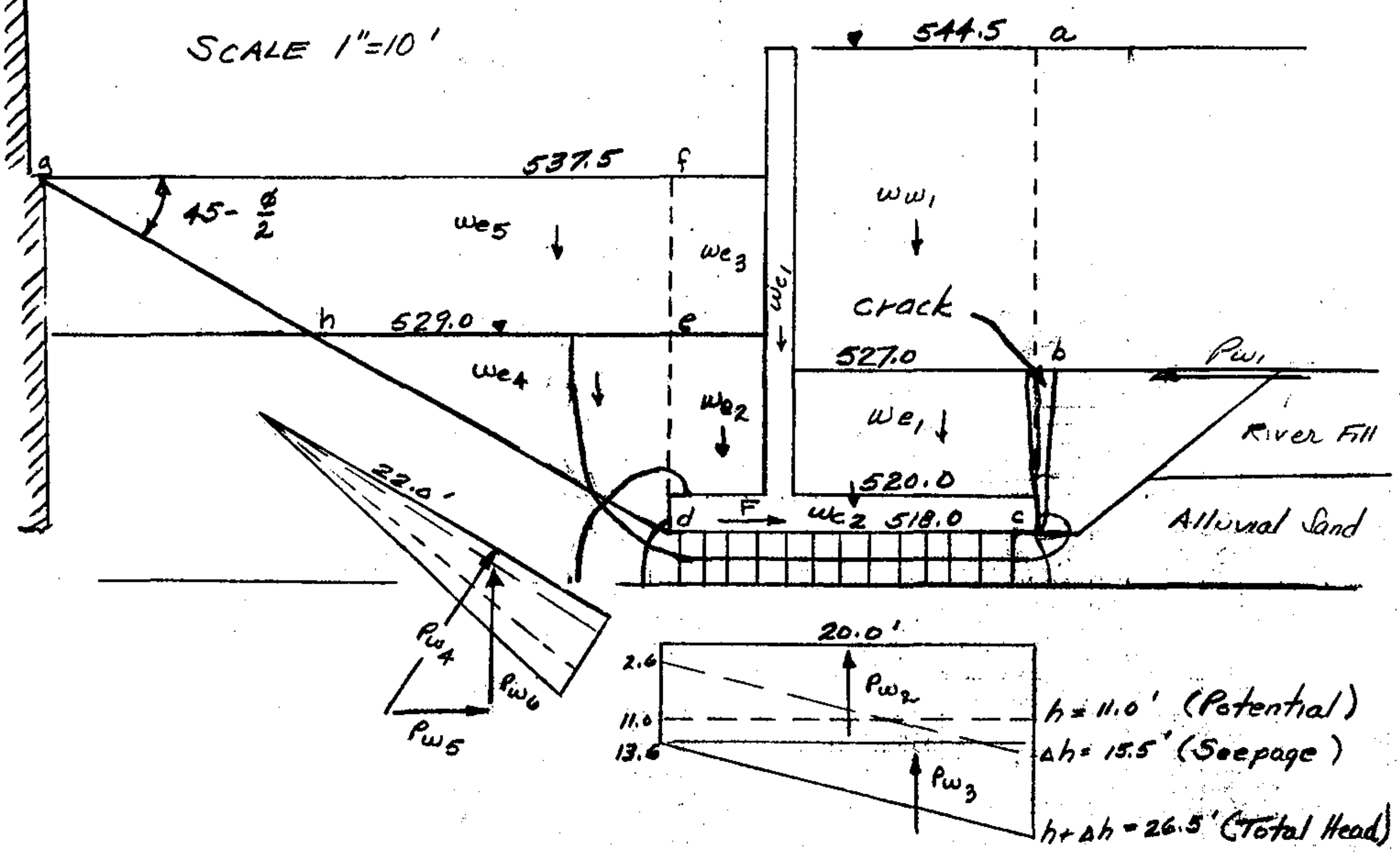
1. Landward Sliding - Flood to Elev. 544.5 and Tailwater at Elev. 529.0
2. Riverward Sliding - Normal Water Levels - Elev 533.0 on both sides

SUBJECT W. WARREN LOCAL PROTECTION - "T" WALL

COMPUTATION Foundation Stability Analysis - Monolith 2

COMPUTED BY ATD CHECKED BY ATD DATE 8 June 61

LOADING No. 1 - Assuming crack in soil over heel to point c



Force		←	↓	↑	↗	
P_{w_1}	26.5 x 26.5 x .5 x 62.5	21,900				
w_{w_1}	13.0 x 17.5 x 62.5		14,200			
W_{e_1}	13.0 x 7.0 x 125		11,400			
W_{e_2}	5.5 x 9.0 x 135		6660			
W_{e_3}	5.5 x 8.5 x 130		6080			
W_{c_1}	1.5 x 24.5 x 150		5520			
W_{c_2}	2.0 x 20.0 x 150		6000			
	Total Weight above c-d	= Σ=	50,860			
P_{w_2}	13.6 x 20.0 x 62.5			17,000		
P_{w_3}	12.9 x 20.0 x .5 x 62.5			8050		
			Σ=	25,050		
W_{e_4}	19.0 x 11.0 x 0.5 x 135		14,100			
W_{e_5}	33.8 x 0.5 x 8.5 x 130		29,000			
		Σ=	43,100			
P_{w_4}	13.6 x 22.0 x 0.5 x 62.5				9350	

SUBJECT W. WARREN, MASS. LOCAL PROTECTION - "T" WALL
COMPUTATION Foundation Stability Analysis - MONOLITH 2
COMPUTED BY afd CHECKED BY afd DATE 8 June 61

LOADING No. 1 Cont.

Sliding Block acdf

$\Sigma V = 50{,}860 - 25{,}050^{\#} = 25{,}810^{\#} \downarrow$

Friction Along c-d $= \Sigma V \tan \phi = 25{,}810 \times .577 = F$
$= 14{,}900^{\#} \rightarrow$

Plane dg - P_{w4} resolved to P_{w5} $(\vec{H})$ + P_{w6} $(V)\uparrow$

$P_{w5} = P_{w4} \sin(45 - \phi/2) = 9350 \times 0.5 = 4675^{\#} \rightarrow$

$P_{w6} = P_{w4} \cos(45 - \phi/2) = 9350 \times 0.866 = 8100^{\#} \uparrow$

Passive Wedge dfg

Effective unit weight of soil $= \dfrac{w_{e4} + w_{e5} - P_{w6}}{\text{Area } dfg} = \bar{\gamma}$

$= \dfrac{43{,}100 - 8100^{\#}}{330 \text{ sf}} = 106.2 \text{ psf/l.f.}$

Passive Pressure $= \frac{1}{2} H^2 \bar{\gamma} \tan^2(45 + \frac{\phi}{2})$

$= \frac{1}{2} \times 19.5^2 \times 106.2 \times 3.0 = P_p = 67{,}200^{\#} \rightarrow$

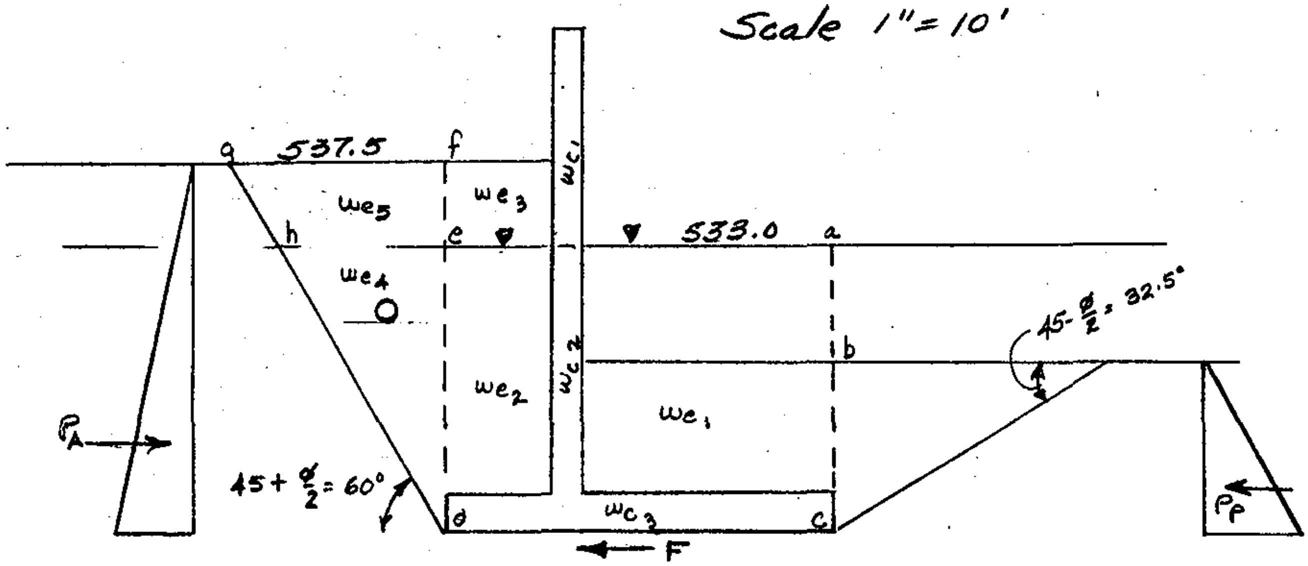
$S.F. = \dfrac{\Sigma \text{ Resisting Forces}}{\Sigma \text{ Driving Forces}} = \dfrac{P_p + F}{P_{w1} - P_{w6}}$

$= \dfrac{67{,}200 + 14{,}900}{21{,}900 - 4680} = \dfrac{12{,}100}{16{,}220} = 5.06$

SUBJECT W. WARREN MASS. LOCAL PROTECTION "T" WALL
COMPUTATION Foundation Stability Analysis - MONOLITH 2
COMPUTED BY afd CHECKED BY afd DATE 14 June 61

LOADING No. 2

Scale 1" = 10'



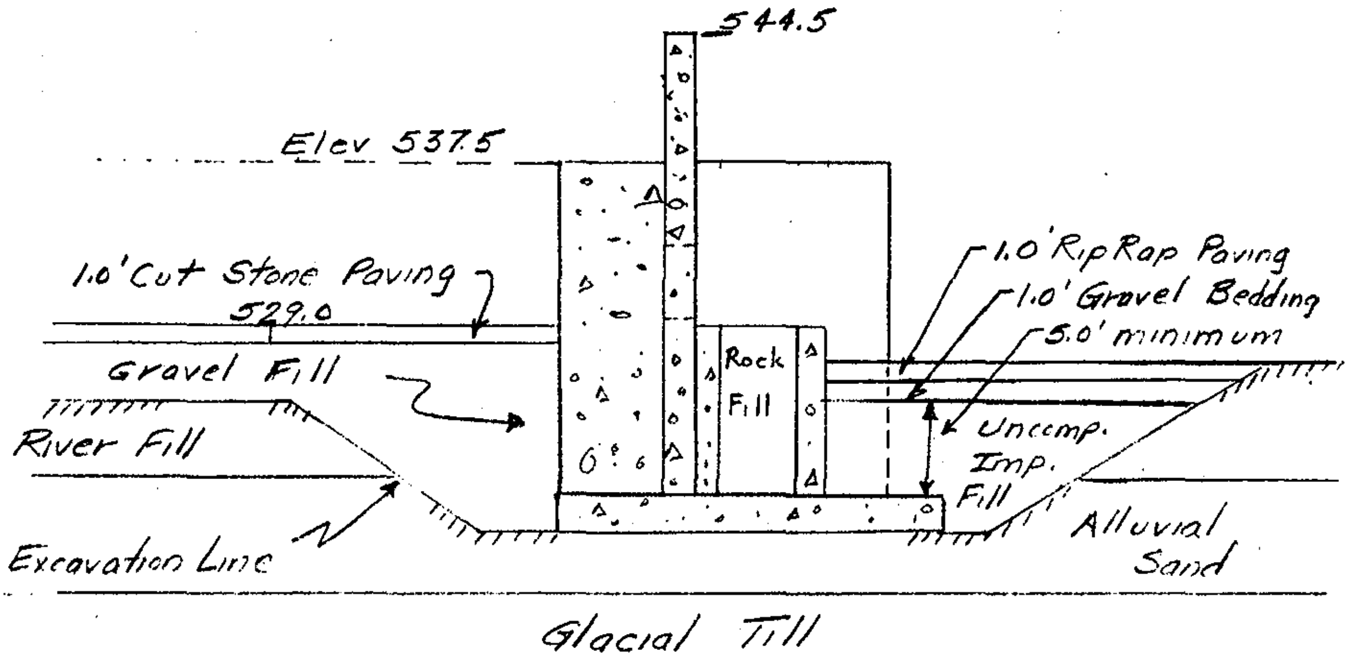
		↓	→	←
w_{c_1}	1.5 × 11.5 × 150	2580		
w_{c_2}	1.5 × 13.0 × 87.5	1710		
w_{c_3}	2.0 × 20.0 × 87.5	3500		
w_{e_1}	7.0 × 13.0 × 62.5	5690		
w_{e_2}	13.0 × 5.5 × 72.5	5180		
w_{e_3}	4.5 × 5.5 × 130	3220		
	Total Weight over c-d = ΣV =	24480		
F	24,480 × 0.577			14,300
w_{e_4}	8.7 × 15.0 × 0.5 × 72.5	4720		
w_{e_5}	19.9 × 0.5 × 4.5 × 130	5805		
	Σ	10,525		
P_A	$0.5 \times 96.5^{a} \times 19.5^{2} \times 0.33^{b}$		6100	
P_P	$0.5 \times 62.5 \times 9.0^{2} \times 2.46^{c}$			6220

a. Effective unit wt. of wedge dfg = $\frac{w_{e_4} + w_{e_5}}{\text{Area } dfg}$ = 96.5 pcf

b. $\tan^2(45 - \frac{30}{2})$ c. $\tan^2(45 + \frac{25}{2})$

$$S.F. = \frac{P_P + F}{P_A} = \frac{6220 + 14,300}{6100} = 3.36$$

Section 3-3 (Plate 5)
Scale 1" = 10'



GATE STRUCTURE MONOLITH ANALYSIS

(Plan shown on Plate 5)

Material Data - Same as page 1

Loading Conditions - Flood to Elev 544.5 and tailwater at Elev. 529.0, landward sliding

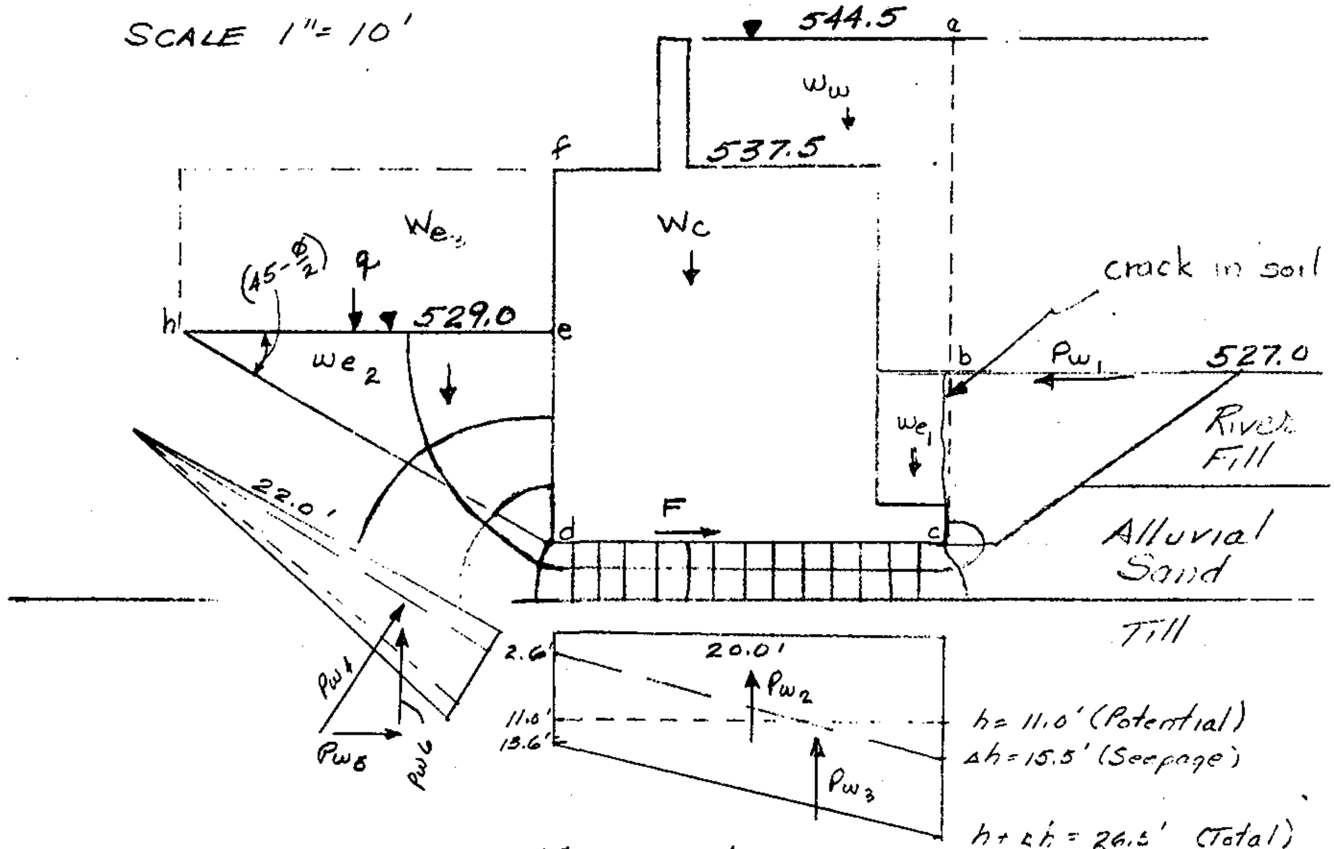
SUBJECT W. WARREN, MASS. LOCAL PROTECTION "T" WALL

COMPUTATION Foundation Stability Analysis - Gate Structure

COMPUTED BY ATL CHECKED BY ATL DATE 14 June 61

LOADING No. 1

SCALE 1" = 10'



BASED ON ENTIRE MONOLITH

		←	↓	↑	↗
P_{w_1}	.5×26.5×26.5×62.5×25	550,000			
W_c	(All Concrete)		449,000		
M	Mechanical Equipment		10,000		
W_w	(All Water)		325,000		
W_{e_1}	(All Earth above c-d)		428,500		
	Total Wt. above c-d = Σ↓		1,212,500		
P_{w_2}	13.6×20×25×62.5			425,000	
P_{w_3}	12.9×20×.5×25×62.5			202,000	
	Total Uplift on c-d = Σ↑		=	627,000	
W_{e_2}	.5×11×18.5×135×25		343,000		
W_{e_3}	6.5×8.5×130×18.5×2 (Surcharge)		265,000		
P_{w_4}	13.6×22.0×.5×25×62.5				234,000

LOADING No. 1 cont.

Sliding Block acdf

$\Sigma V = 1,212,500 - 627,000 = 585,500^{\#} \downarrow$

Friction along c-d = $\Sigma V \tan\phi$ =

$= 585,500 \times 0.577 = \underline{338,000^{\#}} \rightarrow$

Plane dh

P_{w4} resolved to P_{w5} $\overrightarrow{(H)}$ and P_{w6} $(V)\uparrow$

$P_{w5} = P_{w4} \sin(45 - \frac{\phi}{2}) = 234,000 \times 0.500 = \underline{117,000^{\#}} \rightarrow$

$P_{w6} = P_{w4} \cos(45 - \frac{\phi}{2}) = 234,000 \times 0.866 = 202,500^{\#} \uparrow$

Passive Wedge deh

Effective Unit Weight of soil = $\dfrac{We_2 - P_{W6}}{\text{Vol. } deh} = \bar{\gamma}$

$$\bar{\gamma} = \frac{343,000 - 202,500}{18.5 \times 25 \times 11 \times 0.5} = 55.5 \text{ pcf}$$

Effective surcharge distributed over entire plane eh

$$= \frac{We_4}{\text{Area } eh} = \frac{265,000}{25 \times 18.5} = 573 \ \#/ft = q$$

$$\text{Passive Pressure} = \left[\frac{1}{2} H^2 \bar{\gamma} \tan^2(45 - \tfrac{\phi}{2}) + Hq \tan^2(45 + \tfrac{\phi}{2})\right] L$$

$$= \left[\frac{1}{2} \times 11 \times 11 \times 55.5 \times 3.0 + 11 \times 573 \times 3\right] 25$$

$$= \underline{724,000^{\#}} \rightarrow$$

$$S.F. = \frac{\Sigma \text{ Resisting Forces}}{\Sigma \text{ Driving Forces}} = \frac{P_p + F}{P_{w_1} - P_{w_6}}$$

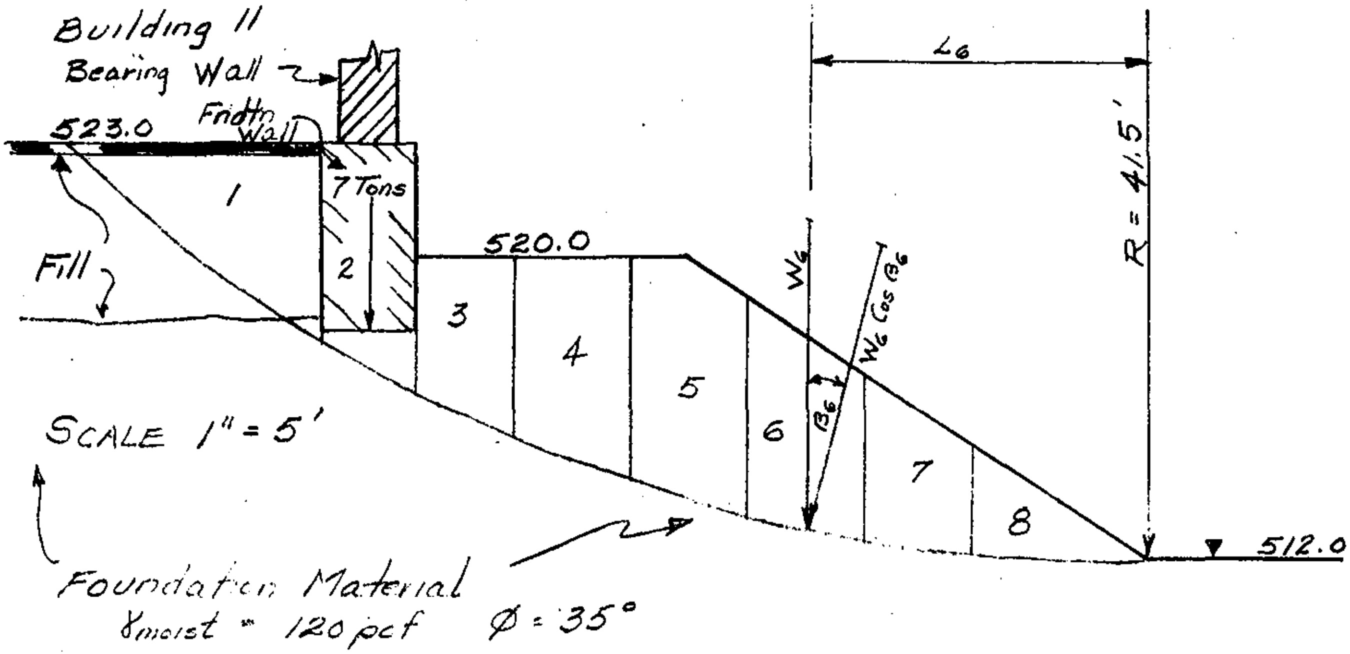
$$= \frac{724{,}000 + 338{,}000}{550{,}000 - 117{,}000} = 2.45$$

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT: W. WARREN, MASS. LOCAL PROTECTION - STA. 9+00 LEFT
COMPUTATION: Building 11 - Foundation Stability Analysis
COMPUTED BY: HHB CHECKED BY: AFS DATE: 6-9 JUNE 1961

CONSTRUCTION CONDITION

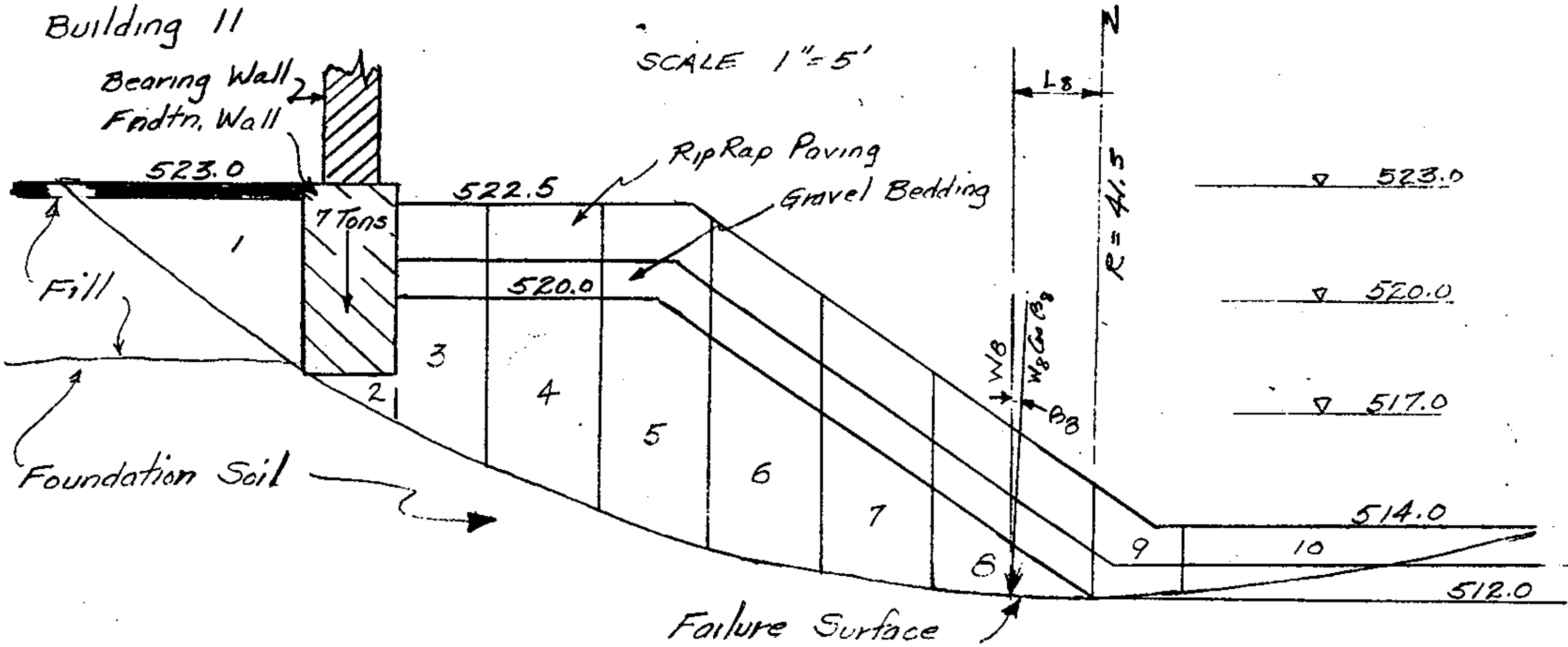


Slice No.	W #	L ft.	WL #-ft	β °	Cos β	W Cos β = N #	R Tan φ ft	N Tan φ R #-ft
1	2010	23.7	47,600	34.8	.820	1,650	29.05	47,800
2	14,210	20.2	288,000	29.2	.872	12,400	29.05	357,000
3	1,200	17.7	21,300	25.2	.904	1,080	29.05	31,500
4	1,870	15.0	28,050	21.1	.932	1,740	29.05	50,600
5	2,160	12.0	25,900	16.7	.957	2,100	29.05	60,000
6	1,800	9.0	16,200	12.4	.975	1,750	29.05	51,000
7	1,300	6.0	7,800	8.4	.983	1,280	29.05	37,200
8	760	3.0	2,280	4.3	.996	756	29.05	22,000

$\Sigma WL = 437,130$ $\Sigma N \tan \phi R = 659,100$

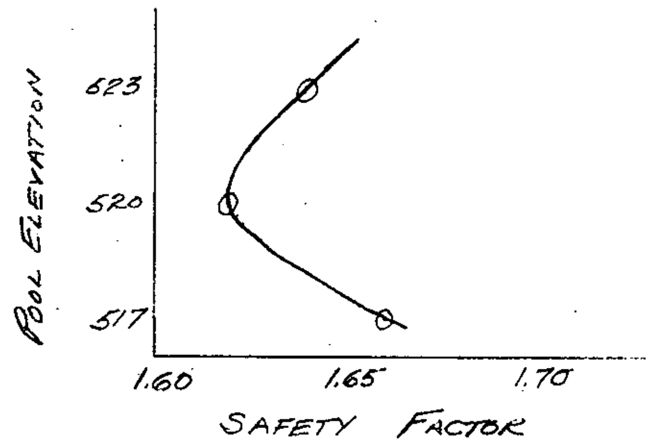
$$S.F. = \frac{\Sigma N \tan \phi R}{\Sigma WL} = \frac{659,100}{437,130} = 1.51$$

OPERATING CONDITION
Critical Pool



All Materials

γ_{moist} = 120 pcf
γ_{sub} = 60 pcf
ϕ = 35°
c = 0 TSF



NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION

CORPS OF ENGINEERS, U.S. ARMY

SUBJECT W. WARREN, MASS. LOCAL PROTECTION - STA 9+00 LEFT

COMPUTATION Building 11 - Foundation Stability Analysis

COMPUTED BY HHB CHECKED BY AFS DATE 6-9 June 61

OPERATING CONDITION

Critical Pool Analysis

Elev. 520.0

$R \tan \phi = 29.05'$

Slice No	W #	L Ft.	WL #-ft	β °	Cos β	W Cos β = N #	N Tan φ R # ft
1	1910	23.7	45300	34.8	.820	1565	45500
2	13,800	20.2	279000	29.2	.872	12020	349000
3	1350	17.7	23900	25.2	.904	1220	35400
4	1840	15.0	27600	21.1	.932	1715	49800
5	2030	12.0	24400	16.7	.957	1942	56500
6	1640	9.0	14800	12.4	.975	1600	46400
7	1220	6.0	7300	8.4	.988	1205	35000
8	1240	2.4	2980	4.3	.996	1236	35800
9	345	-1.2	-414	1.8	1.000	345	10000
10	620	-5.8	-3600	9.0	.988	613	17800

$\Sigma WL = 421\,266$ $\qquad$ $\Sigma N \tan \phi R = 681\,200$

$$SF = \frac{681,200}{421,266} = 1.62$$

SUBJECT QUABOAG RIVER, WEST WARREN, MASS.
COMPUTATION DESIGN ASSUMPTIONS
COMPUTED BY C.C.C. CHECKED BY R.A.K. DATE 6-23-61

"T" SECTION DESIGN

Fill on land and river sides:	γ moist = 130. pcf. γ sat. = 135 " γ sub. = 72.5 " ϕ = 25° $\tan^2(45° - \frac{\phi}{2}) = .406$	These preliminary values were used for the "T" Sect. but are superseded by the values below.

Loading No. 1 : Flood to El. 544.5 = Creep path b to e.
" 2 : " " " b to d.
" 3 : " 541.5 " b to e
" 4 : " " " b to d
" 5 : Low Water : 529.0

GATE MONOLITH

Fill, River Side

γ moist = 120. pcf.
γ Sat = 125 "
γ Sub. = 62.5 "
ϕ = 25°
$\tan^2(45° - \frac{\phi}{2}) = .406$

Fill, Land Side

γ moist = 130. pcf.
γ sat. = 135 "
γ sub. = 72.5 "
ϕ = 30°
$\tan^2(45° - \frac{\phi}{2}) = 1/3$

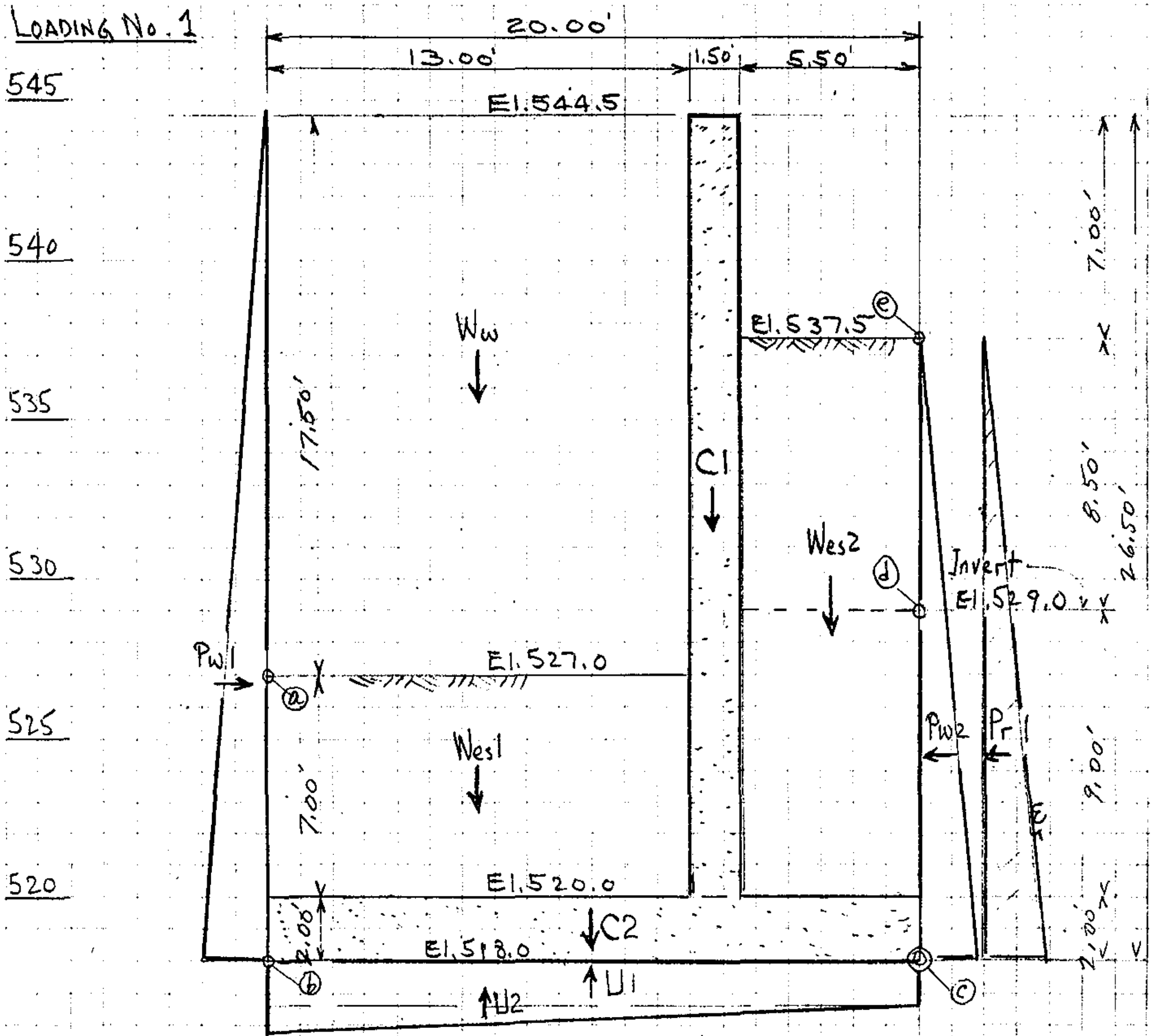
Max. allowable base press. : 3,000 psf.
Concrete : 150 pcf.
Water : 62.5 "

f'_c = 3,000. psi.
f_c = 1050 psi
f_s = 20,000 psi.
v = 90 psi.
u = 300/210 psi.

j = .868
k = 160.
a = 1.475

SUBJECT Quaboag River, West Warren, Mass.
COMPUTATION "T" Section Design
COMPUTED BY CCC CHECKED BY R. A. K. DATE 5-12-61

Loading No. 1

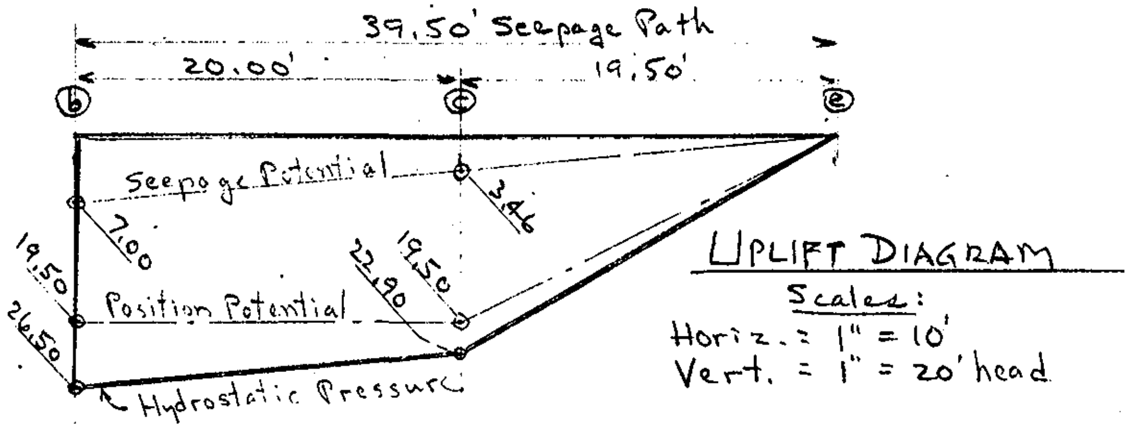


Section C-C

Scale = □ 1'
1'

SUBJECT QUABOAG RIVER, WEST WARREN, MASS.
COMPUTATION "T" SECTION DESIGN
COMPUTED BY CCC CHECKED BY R.A.K. DATE 5-12-61

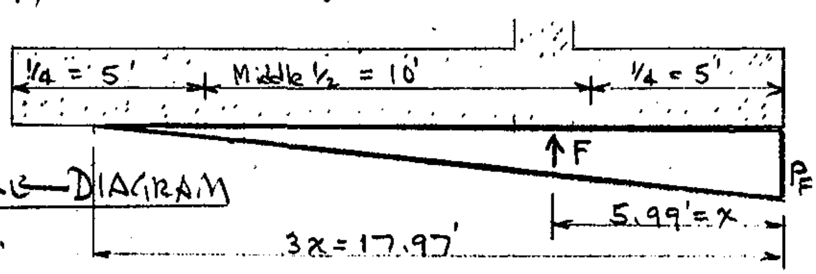
LOADING No. 1



	$\Sigma M_c = 0$	↓	↑	→	←	ARM	↻	↺
C1	1.5 × 24.5 × 150.	5,513.				6.25		34,453.
C2	20.0 × 2.0 × 150.	6,000.				10.0		60,000.
Ww	13.0 × 17.5 × 62.5	14,219.				13.5		191,953.
Wes1	13.0 × 7.0 × 135.	12,285.				13.5		165,848.
Wes2	5.5 × 17.5 × 135.	12,994.				2.75		35,733.
U1	20.0 × 22.9 × 62.5		28,625.			10.0	286,250	
U2	0.5 × 20.0 × 3.60 × 62.5		2,250.			13.333	29,999.	
F			① 20,136.			⑤ 5.99.	④ 120,535.	
Pw1	0.5 × 26.5 × 26.5 × 62.5			21,945		8.833	193,843.	
Pw2	0.5 × 19.5 × 22.90 × 62.5				13,955.	6.50		90,705.
Pr1					② 7,990.	6.50		③ 51,935.
		51,011.	51,011.	21,945	21,945		630,627.	630,627.

$$w_r = \frac{P_r 1}{0.5(19.5)^2} = \frac{7990}{0.5 \times 380.25} = \underline{42.0 \text{ psf increment.}}$$

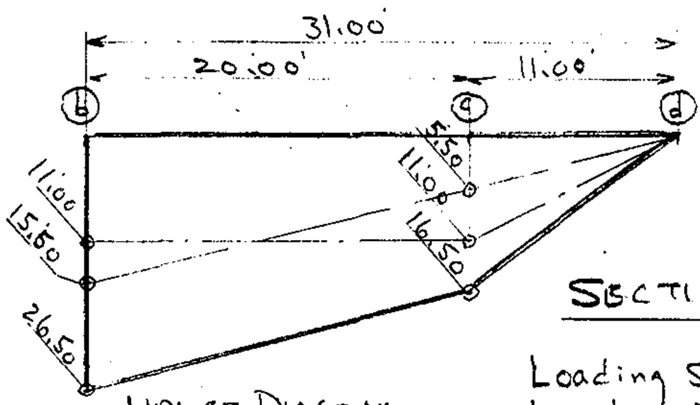
$$P_F = \frac{2F}{3x} = \frac{2 \times 20,136}{17.97} = \underline{2,241. \text{ psf.}}$$



BASE PRESSURE DIAGRAM
Scale = 1' □ 1'

SUBJECT Quaboag River, West Warren, Mass.
COMPUTATION "T" Section Design
COMPUTED BY CCC CHECKED BY R.A.K. DATE 5-15-61

Loading No. 2

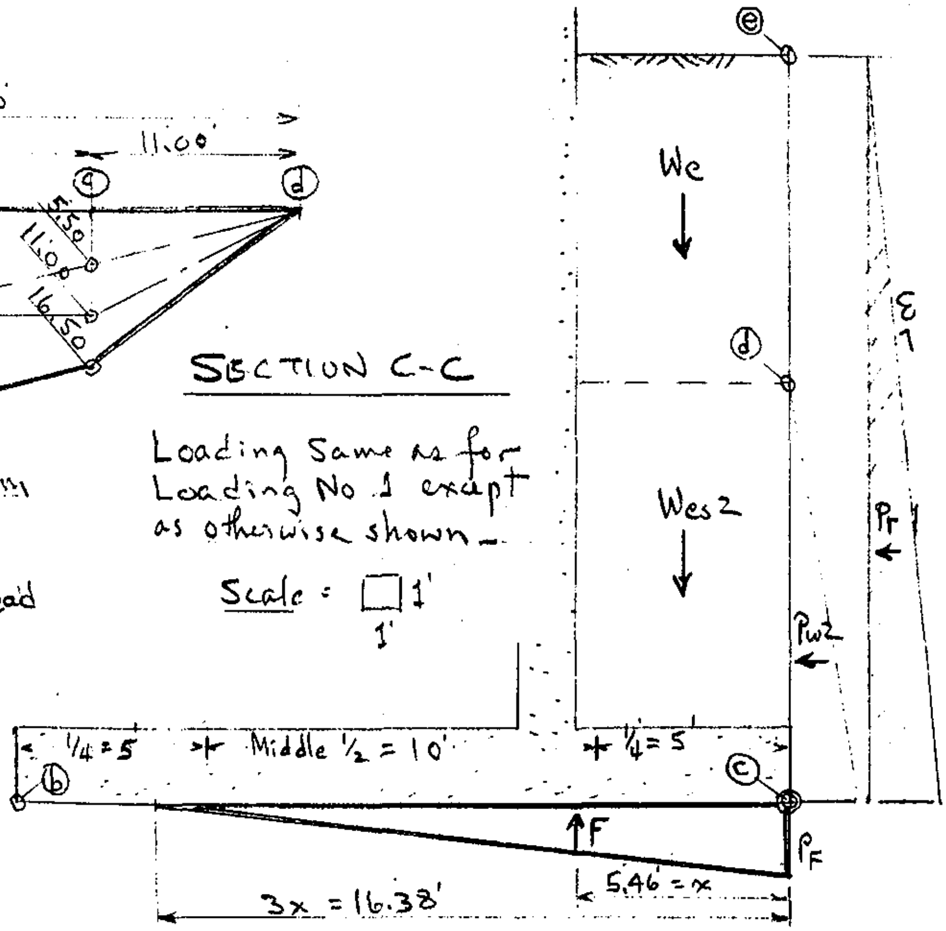


Uplift Diagram
Scales:
Horiz. 1" = 10'
Vert. : 1" = 20' head

SECTION C-C

Loading same as for Loading No 1 except as otherwise shown.

Scale: 1'



$$W_r = \frac{P_r 1}{0.5\,(19.5)^2} = \frac{16{,}273}{0.5 \times 380.25} = \underline{85.6 \text{ psf increment}}$$

$$P_F = \frac{2F}{3X} = \frac{2 \times 23{,}903}{16.38} = \underline{2{,}919 \text{ psf}}$$

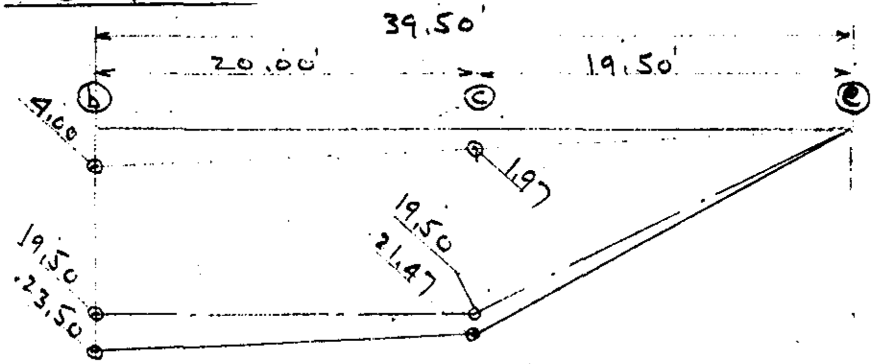
	ΣMⓒ = 0	↓	↑	→	←	Arm	↷	↶
C1		5,513						34,453
C2		6,000						60,000
Ww		14,219						191,953
Wes1		12,285						165,848
We	5.5 × 8.5 × 130	6,078				2.75		16,713
Wes2	5.5 × 9.0 × 135	6,683				2.75		18,377
U1	20.0 × 16.5 × 62.5		20,625			10.0	206,250	
U2	0.5 × 20.0 × 10.0 × 62.5		6,250			13,333	83,331	
F			① 23,903			⑤ 5.46	④ 130,494	
Pw1				21,945			193,843	
Pw2	0.5 × 11.0 × 16.5 × 62.5				5,672	3.667		20,799
Pr1					② 16,273	6.50		③ 105,775
		50,778	50,778	21,945	21,945		613,918	613,918

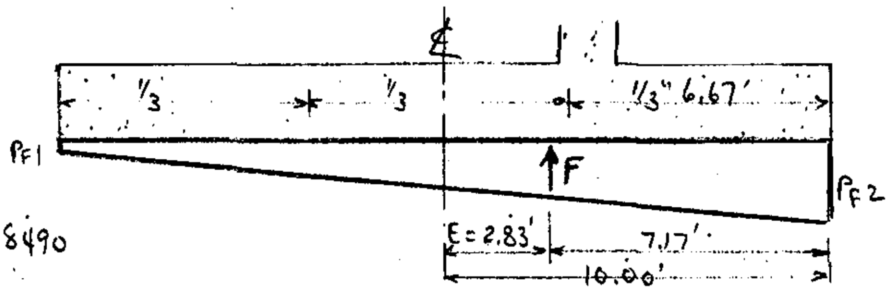
NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT: QUABOAG RIVER, WEST WARREN, MASS.
COMPUTATION: "T" SECTION DESIGN
COMPUTED BY: C.F.C. CHECKED BY: R.A.K. DATE: 5-15-61

LOADING NO. 3





$F/L = \frac{20,466}{20} = 1,023.3$

$6E/L = 6 \times 2.83/20 = 0.8490$

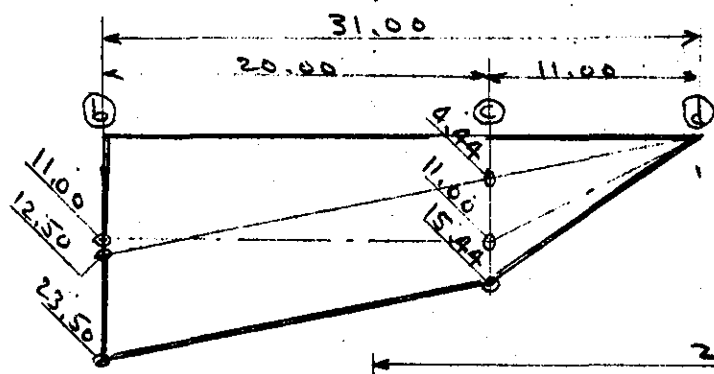
$P_{F1}, P_{F2} = \frac{F}{L}(1 \pm 6E/L) = \underline{155. psf.}, \underline{1892. psf}$

$w_r = \frac{Pr1}{0.5(19.5)^2} = \frac{4175}{0.5 \times 380.25} = \underline{22.0 \text{ psf}/1}$

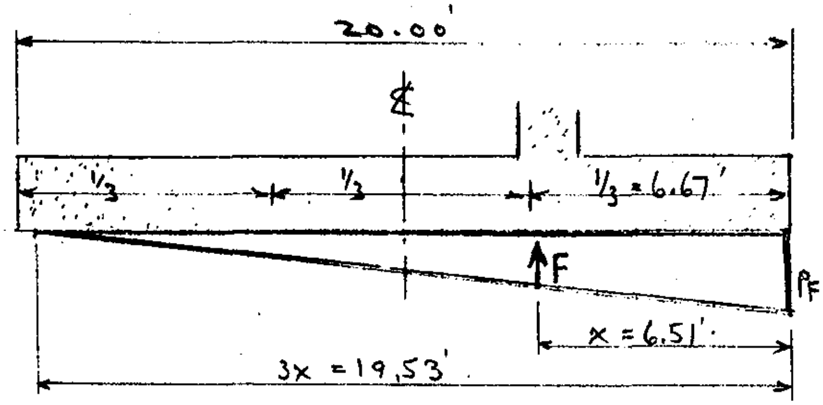
	ΣMc = 0	↓	↑	→	←	Arm	↷	↶
C1		5,513						34,453
C2		6,000						60,000
Ww	13.0×14.5×62.5	11,781				13.5		159,047
Wes1		17,285						165,848
Wes2		12,994						35,733
U1	20.0×21.47×62.5		26,838			10.0	268,375	
U2	0.5×20.0×2.03×62.5		1,269			13.333	16,917	
F			① 20,466			⑤ 7.17	④ 146,788	
Pw1	0.5×23.5×23.5×62.5			17,258		7.833	135,180	
Pw2	0.5×19.5×21.47×62.5				13,083	6.5		85,041
Pr1					② 4,175	6.5		③ 27,138
		48,573	48,573	17,258	17,258		567,260	567,260

SUBJECT Quaboag River, West Warren, Mass.
COMPUTATION "T" Section Design
COMPUTED BY CCC. CHECKED BY RAK. DATE 5-15-61

Loading No. 4



Uplift Diagram
Scales:
Horiz. : 1" = 10'
Vert. : 1" = 20' head



Base Pressure Diagram

Scale: □ 1' × 1'

$$w_r = \frac{P_r 1}{0.5(19.5)^2} = \frac{11{,}950}{0.5 \times 380.25} = \underline{62.9 \text{ p.s.f. increment}}.$$

$$P_F = \frac{2F}{3x} = \frac{2 \times 24{,}002}{19.53} = \underline{2{,}458. \text{ psf}}.$$

		↓	↑	→	←	Arm	↷	↶
C1		5,513						34,453
C2		6,000						60,000
Ww		11,781						159,047
Wes1		12,285						165,848
We		6,078						16,713
Wes2		6,683						18,377
U1	20.0×15.44×62.5		19,300			10.0	193,000	
U2	0.5×20.0×8.06×62.5		5,038			13.333	67,165	
F			① 24,002			⑤ 6.51	④ 156,231	
Pw1				17,258			135,180	
Pw2	0.5×11.0×15.44×62.5				5,308	3.667		19,463
Pr1					② 11,950	6.5		③ 77,675
		48,340	48,340	17,258	17,258		551,576	551,576

SUBJECT QUABOAG RIVER, WEST WARREN, MASS.
COMPUTATION "T" SECTION DESIGN
COMPUTED BY G.C.C. CHECKED BY R.A.K. DATE 5-15-61

LOADING NO. 5

$\frac{F}{L} = \frac{24,434}{20} = 1222.$

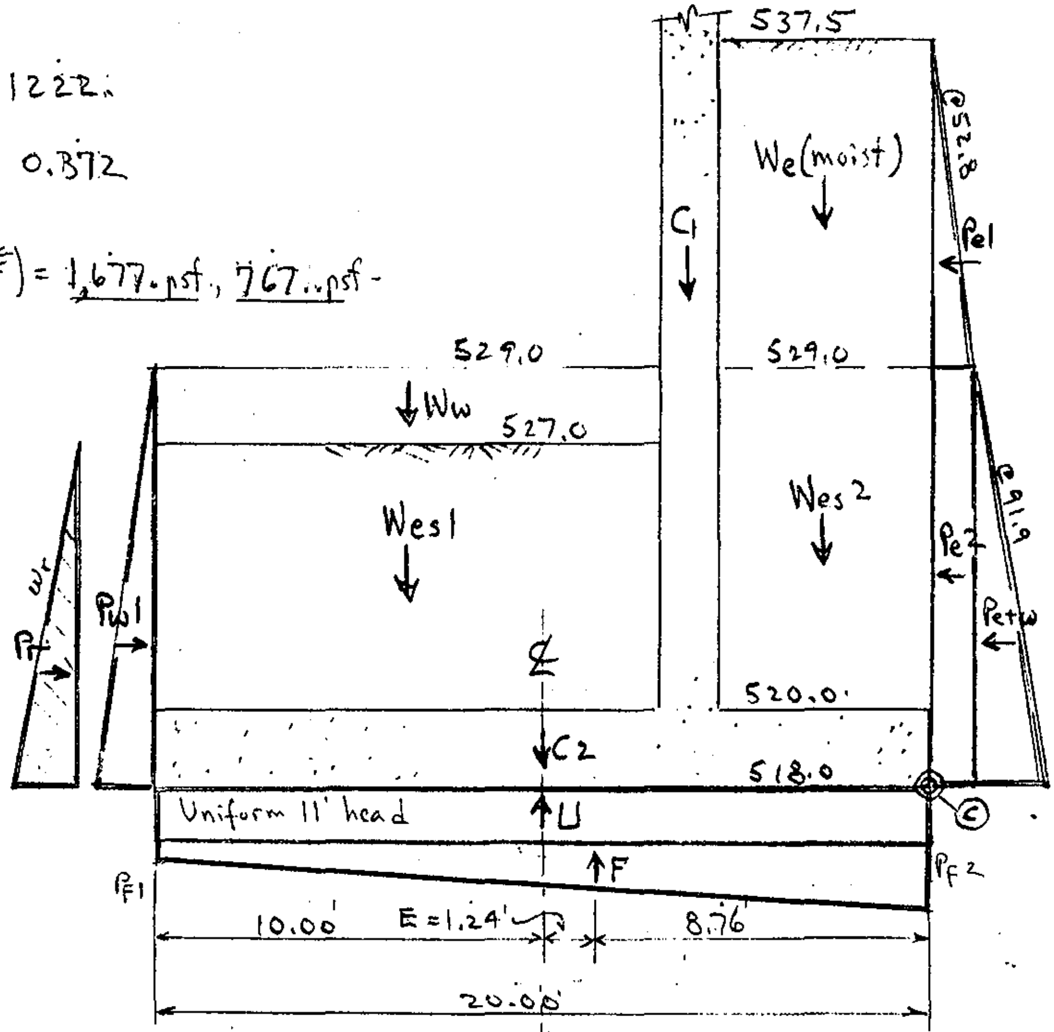
$\frac{6E}{L} = \frac{6 \times 1.24}{20} = 0.372$

$P_{F2}, P_{F1} = \frac{F}{L}\left(1 \pm \frac{6E}{L}\right) = 1,677.\text{psf}, 767.\text{psf}$

$w_r = \frac{P_r}{0.5(9.0)^2}$

$= \frac{8623.}{0.5 \times 81}$

$= 213.\text{psf}/1$



SECTION C-C Scale: □1' 1'

	$\Sigma M_c = 0$	↓	↑	→	←	ARM	↻	↺
C1		5,513.						34,453.
C2		6,000.						60,000.
Ww	13.0×2.0×62.5	1,625.				13.5		21,937.
Wes1		12,285.						165,848.
We		6,078.						16,713.
Wes2		6,683.						18,377.
U	20.0×11.0×62.5		13,750			10.0	137,500.	
F			① 24,434.			⑤ 8,759	④ 214,018	
Pw1	0.5×11.0×11.0×62.5			3,781.		3.667	13,866.	
Pr				② 8,623.		3.0	③ 25,869.	
Pe1	0.5×8.5×8.5×52.8				1,907.	13.833		26,385.
Pe2	11.0×8.5×52.8				4,937.	5.5		27,152.
Pe+w	0.5×11.0×11.0×91.9				5,560	3.667		20,388.
		38,184	38,184	12,404	12,404		391,253	391,253

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Quaboag River, West Warren, Mass.
COMPUTATION "T" Section Design
COMPUTED BY C.C.C. CHECKED BY R.H.K. DATE 6-14-61

Stem Moments for Loading No. 2

		→	←	Arm	↻	↺
	El. 537.5					
Pw1	$0.5(7.00)^2\,62.5$	1531. = V		2.333	3572. = M	
	El. 533.5					
Pw1	$0.5(11.00)^2\,62.5$	3781.		3.667	13,866.	
Pr1	$0.5(4.00)^2\,85.6$		685.	1.333		913.
	V =	3096.		M =	12,953.	
	El. 529.5					
Pw1	$0.5(15.00)^2\,62.5$	7,031.		5.0	35,156.	
Pr1	$0.5(8.00)^2\,85.6$		2739.	2.667		7,305.
	V =	4,292.		M =	27,851.	
	El. 526.5					
Pw1	$0.5(18.00)^2\,62.5$	10,125.		6.0	60,750.	
Pr1	$0.5(11.00)^2\,85.6$		5,179.	3.667		18,991.
Pw2	$0.5(2.50)^2\,62.5$		195.	0.833		163.
		10,125.	5,374.		60,750.	19,154.
	V =	4,751.		M =	41,596.	
	El. 523.5					
Pw1	$0.5(21.00)^2\,62.5$	13,781.		7.00	96,469.	
Pes1	$0.5(3.50)^2\,29.4$	180.		1.167	210.	
Pr1	$0.5(14.00)^2\,85.6$		8389.	4.667		39,151.
Pw2	$0.5(5.50)^2\,62.5$		945.	1.833		1733.
		13,961.	9334.		96,679.	40,884.
	V =	4,627.		M =	55,795.	
	El. 520.5					
Pw1	$0.5(24.00)^2\,62.5$	18,000.		8.0	144,000.	
Pes1	$0.5(6.50)^2\,29.4$	621.		2.167	1,346.	
Pr1	$0.5(17.00)^2\,85.6$		12,369.	5.667		70,096.
Pw2	$0.5(8.50)^2\,62.5$		2258.	2.833		6,396.
		18,621.	14,627.		145,346.	76,492.
	V =	3,994.		M =	68,854.	

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT QUABOAG RIVER, WEST WARREN, MASS.
COMPUTATION "T" SECTION DESIGN
COMPUTED BY C.C.C. CHECKED BY R.A.K. DATE 6-15-61

Stem Moments for Loading No. 5

		→	←	Arm	↷	↶
	El. 533.5					
Pe1	$0.5(4.00)^2\ 52.8$		V = 422.	1.333		M = 563.
	El. 529.5					
Pe1	$0.5(8.00)^2\ 52.8$		V = 1,690.	2.667		M = 4,506.
	El. 526.5					
Pw1	$0.5(2.50)^2\ 62.5$	195.		0.833	163.	
Pe1	$0.5(8.5)^2\ 52.8$		1,907.	5.333		10,172.
Pe2	$2.5\times8.5\times52.8$		1,122.	1.25		1,403.
Pew	$0.5(2.5)^2\ 91.9$		287.	0.833		239.
		195.	3,316.		163.	11,814.
			V = 3,121.			M = 11,651.
	El. 523.5					
Pw1	$0.5(5.50)^2\ 62.5$	945.		1.833	1,733.	
Pr	$0.5(3.50)^2\ 213.$	1,305.		1.167	1,522.	
Pe1			1907.	8.333		15,891.
Pe2	$5.5\times8.50\times52.8$		2,468.	2.75		6,788.
Pew	$0.5(5.5)^2\ 91.9$		1,390.	1.833		2,548.
		2,250.	5765.		3,255.	25,227.
			V = 3,515.			M = 21,972.
	El. 520.5					
Pw1	$0.5(8.50)^2\ 62.5$	2258.		2.833	6,396.	
Pr	$0.5(6.50)^2\ 213.$	4500.		2.167	9,751.	
Pe1			1,907.	11.333		21,612.
Pe2	$8.50\times8.50\times52.8$		3,815.	4.25		16,213.
Pew	$0.5(8.50)^2\ 91.9$		3,320.	2.833		9,405.
		6758.	9042.		16,147.	47,230.
			V = 2284.			M = 31,083.

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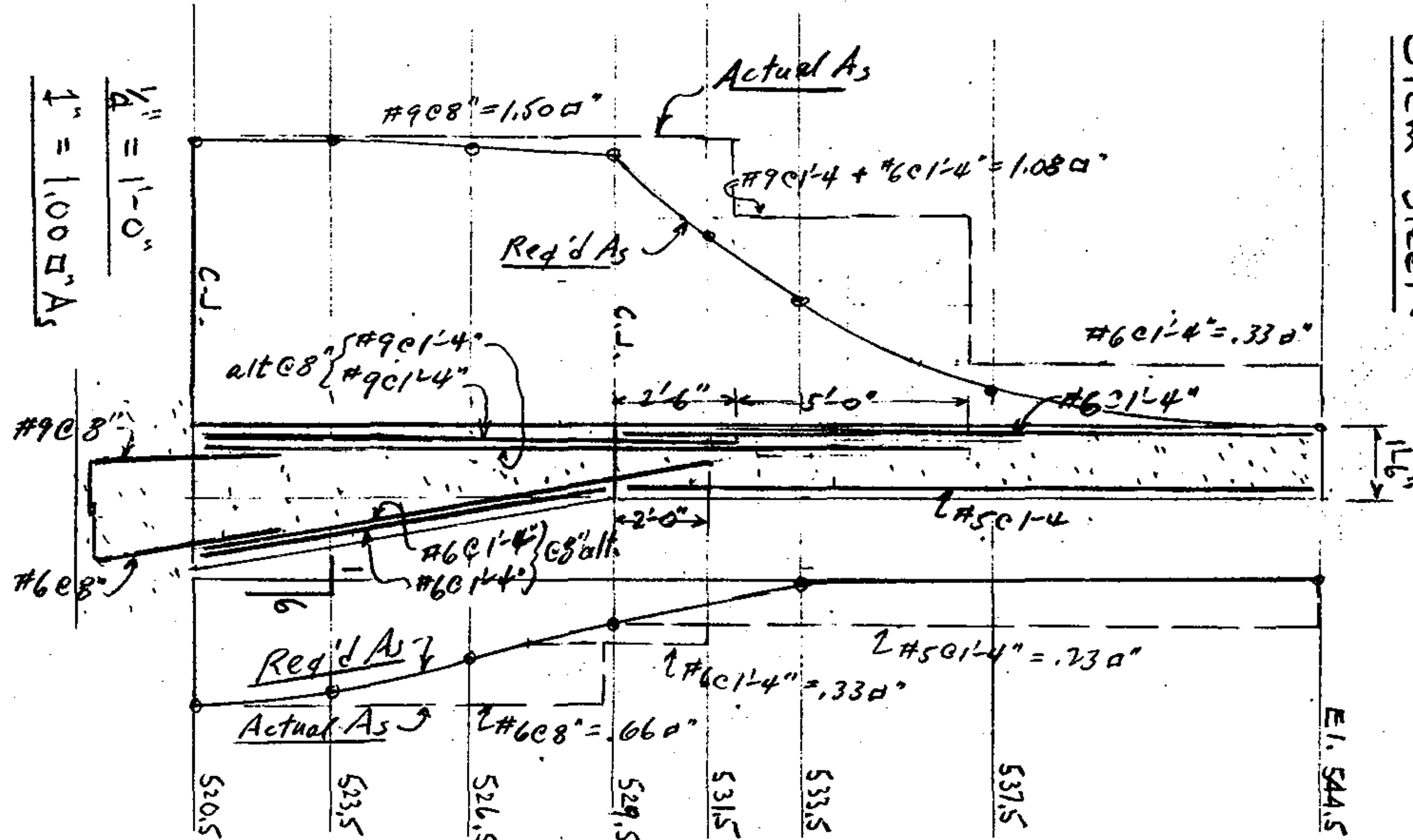
NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Quaboag River, West Warren, Mass.
COMPUTATION "T" Section Design
COMPUTED BY C.C.C. CHECKED BY R.A.K. DATE 6-15-61
PAGE 21

Stem Steel:



$b = 12"$
$j = .885$
$K = 160$
$a = 1.475$
$d = t - 4.5"$

$$v = \frac{V}{bjd} = .09416\frac{V}{d}$$

$$Reqd.\ d = \sqrt{\frac{M^{K'}}{.160}} = 2.5\sqrt{M^{K'}}$$

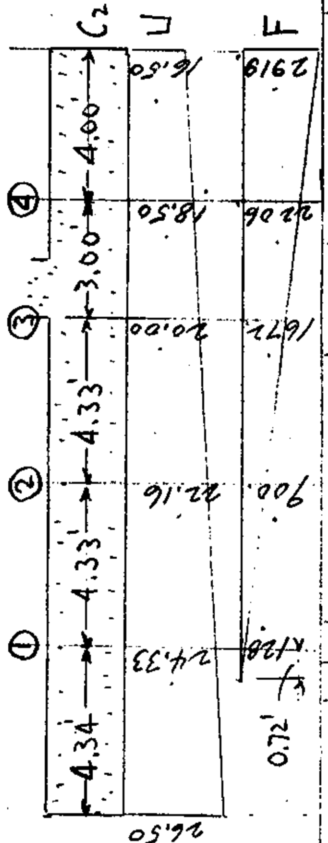
$$Reqd\ A_s = \frac{M^{K'}}{a\,d} = .6780\,\frac{M^{K'}}{d}$$

$$u = \frac{V}{\Sigma_o jd}$$

	LEFT FACE (Loading No. 2)						RIGHT FACE (Loading No. 5)				
Elev.	520.5	523.5	526.5	529.5	533.5	537.5	520.5	523.5	526.5	529.5	533.5
t"	36.	30.	24.	18.	18.	18.	36.	30.	24.	18.	18.
d"	31.5	25.5	19.5	$\boxed{13.5}$	13.5	13.5	31.5	25.5	19.5	13.5	13.5
V#	3,994	4,627	4,751	4,292	3,096	1,531	2284	3,515	3,121	1,690	422
v	11.9	17.1	22.9	29.9	—	—	—	—	—	—	—
M#'	68,854	55,795	41,596	27,851	12,953	3,572	31,083	21,972	11,651	4,506	563
Req'd d"	20.8	18.7	16.1	$\boxed{13.2}$	9.0	4.7	14.0	11.7	8.5	5.3	—
Req'd As	1.48	1.48	1.45	1.40	0.65	0.18	0.67	0.58	0.41	0.23	0.03
Use	#9@8" →				#9+6@1'-4"	#6@1'-4"	#6@8" →			#6@1-4	#5@1-4
Actual As	1.50 →				1.08	0.33	0.66 →			0.33	0.23
Σo	5.3 →				4.5	1.8	3.5 →			1.8	1.5
u	27.0	38.6	51.8	67.5	57.7	71.2	23.4	44.5	51.7	78.8	23.6

SUBJECT Quaboag River, West Warren, Mass.
COMPUTATION "T" Section Design
COMPUTED BY C.C.C. CHECKED BY R.A.K. DATE 6-16-61

Base Moments for Loading No. 2



	Sec. ①	↓	↑	Arm	↷	↶
Ww	4.34 × 17.5 × 62.5	4,747.				
Wes	4.34 × 7.0 × 135	4,101.		2.17		22,026.
C2	4.34 × 2.0 × 150	1,302				
U1	4.34 × 24.33 × 62.5		6,600.	2.17	14,321.	
U2	0.5 × 4.34 × 2.17 × 62.5		294.	2.893	851.	
F	0.5 × 0.72 × 128.		46.	0.24	11.	
		10,150.	6,940.		15,183.	22,026.
		V = 3,210.				M = 6,843.
	Sec. ②					
Ww	8.67 × 17.5 × 62.5	9,483				
Wes	8.67 × 7.0 × 135.	8,193.		4.335		87,901.
C2	8.67 × 2.0 × 150.	2,601				
U1	8.67 × 22.16 × 62.5		12,008.	4.335	52,054.	
U2	0.5 × 8.67 × 4.34 × 62.5		1,176.	5.78	6,797.	
F	0.5 × 5.05 × 900.		2,273.	1.683	3,825.	
		20,277	15,457		62,676.	87,901.
		V = 4,820				M = 25,225.
	Sec. ③					
Ww	13.0 × 17.5 × 62.5	14,219				
Wes	13.0 × 7.0 × 135.	12,285		6.5		197,626.
C2	13.0 × 2.0 × 150.	3,900				
U1	13.0 × 20.0 × 62.5		16,250.	6.5	105,625.	
U2	0.5 × 13.0 × 6.5 × 62.5		2,641.	8.667	22,886.	
F	0.5 × 9.38 × 1672.		7,842.	3.127	24,521.	
		30,404	26,733		153,032.	197,626.
		V = 3,671.				M = 44,594.
	Sec. ④					
We	4.0 × 8.5 × 130.	4,420				
Wes2	4.0 × 9.0 × 135	4,860.		2.0	20,960.	
C2	4.0 × 2.0 × 150	1,200				
U1	4.0 × 16.50 × 62.5		4,125.	2.0		8,250.
U2	0.5 × 4.0 × 2.0 × 62.5		250.	1.333		333
F1	4.0 × 2206.		8,824	2.0		17,648.
F2	0.5 × 4.0 × 713.		1,426.	2.667		3,803.
		10,480	14,625.		20,960.	30,034.
		V = 4,145.				M = 9,074.

NED FORM 223
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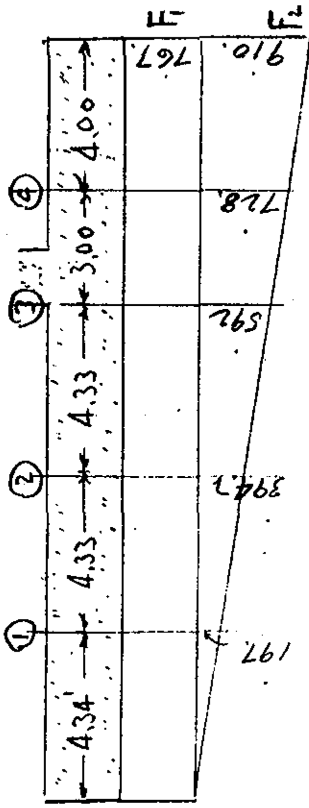
NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT QUABOAG RIVER, WEST WARREN, MASS.
COMPUTATION "T" SECTION DESIGN
COMPUTED BY C.C.C. CHECKED BY R.A.K. DATE 6-16-61

Base Moments for Loading No. 5



	Sec. ①	↓	↑	Arm	↷	↶
Ww	4.34×2.0×62.5	543.				
Wes1	4.34×7.0×135	4,101.		2.17		12,903.
C2	4.34×2.0×150	1,302.				
U	4.34×11.0×62.5		2,984.	2.17	6,475.	
F1	4.34×767.		3,329.	2.17	7,223.	
F2	0.5×4.34×197.		427.	1.447	619.	
		5,946.	6,740.		14,317.	12,903.
		V =	794.	M =	1,414.	
	Sec. ②					
Ww	8.67×2.0×62.5	1,084				
Wes1	8.67×7.0×135.	8,193		4.335		51,491.
C2	8.67×2.0×150.	2,601.				
U	8.67×11.0×62.5		5,961.	4.335	25,841.	
F1	8.67×767.		6,650.	4.335	28,827.	
F2	0.5×8.67×394.		1,708.	2.89	4,936.	
		11,878	14,319.		59,604.	51,491.
		V =	2,441.	M =	8,113.	
	Sec. ③					
Ww	13.0×2.0×62.5	1,625.				
Wes1	13.0×7.0×135.	12,285.		6.5		115,765.
C2	13.0×2.0×150.	3,900.				
U	13.0×11.0×62.5		8,938.	6.5	58,094.	
F1	13.0×767.		9,971.	6.5	64,812.	
F2	0.5×13.0×592.		3,848.	4.333	16,673.	
		17,810.	22,757.		139,579.	115,765.
		V =	4,947.	M =	23,814.	
	Sec. ④					
We	4.0×8.5×130.	4,420.				
Wes2	4.0×9.0×135	4,860		2.0	20,960.	
C2	4.0×2.0×150	1,200				
U	4.0×11.0×62.5		2,750.	2.0		5,500
F1	4.0×1495.		5,980.	2.0		11,960
F2	0.5×4.0×182.		364.	2.667		971.
		10,480.	9094.		20,960.	18,431.
	V =	1,386.		M =	2,529.	

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Quaboag River, West Warren, Mass.
COMPUTATION "T" Section Design
COMPUTED BY C.C.F. CHECKED BY R.A.K. DATE 6-12-61

	Sec. (3), Loading No. 1	↓	↑	→	←	Arm	↻	↺
Ww		14,219				6.50		92,424
Wesl		12,285				6.50		79,852
C2	13.0 × 2.0 × 150	3,900				6.50		25,350
U1	13.0 × 24.16 × 62.5		19,630			6.50	127,595	
U2	0.5 × 13.0 × 2.34 × 62.5		951			8.667	8,239	
F	0.5 × 10.97 × 1368		7,503			3.657	27,440	
		30,404	28,084				163,274	197,626
		V = 2,320						M = 34,352

	Sec. (3), Loading No. 2	↓	↑	→	←	Arm	↻	↺
Ww		14,219						92,424
Wesl		12,285						79,852
C2		3,900						25,350
U1	13.0 × 20.0 × 62.5		16,250			6.5	105,625	
U2	0.5 × 13.0 × 6.50 × 62.5		2,641			8.667	22,886	
F	0.5 × 9.38 × 1672		7,840			3.127	24,514	
		30,404	26,731				153,025	197,626
→		V = 3,673		(This Loading Governs)				M = 44,601

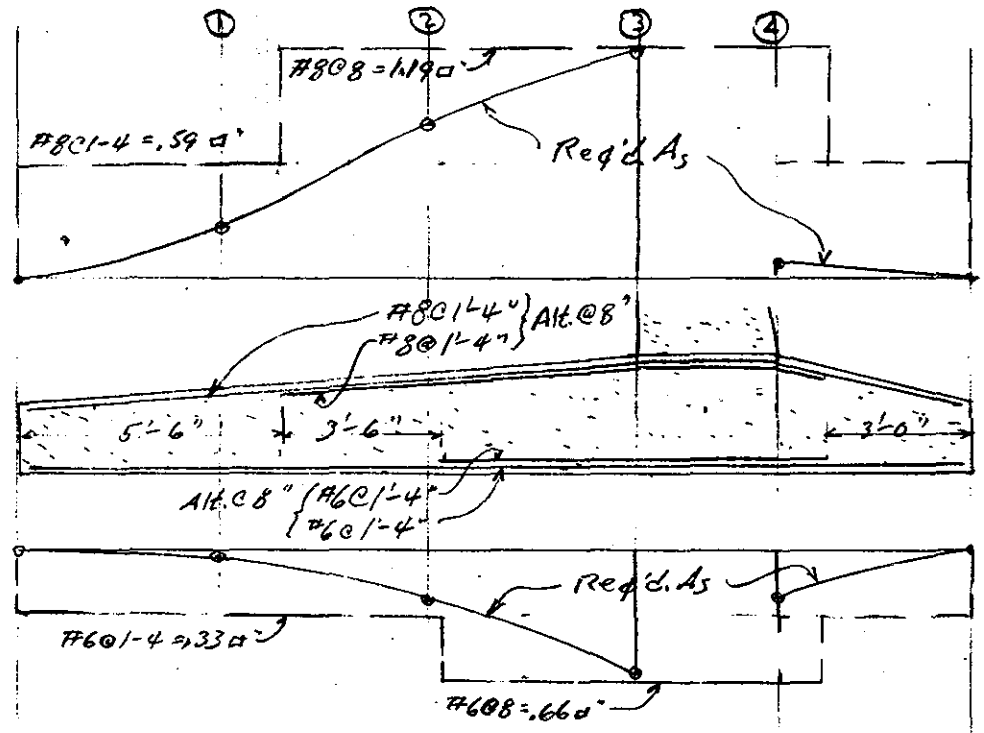
	Sec. (3), Loading No. 3	↓	↑	→	←	Arm	↻	↺
Ww		11,781				6.50		76,577
Wesl		12,285						79,852
C2		3,900						25,350
U1	13.0 × 22.18 × 62.5		18,021			6.50	117,138	
U2	0.5 × 13.0 × 1.32 × 62.5		536			8.667	4,648	
F1	13.0 × 155		2,015			6.50	13,098	
F2	0.5 × 13.0 × 1129		7,339			4.333	31,798	
		27,966	27,911				166,682	181,779
		V = 55						M = 15,097

	Sec. (3), Loading No. 4	↓	↑	→	←	Arm	↻	↺
Ww		11,781						76,577
Wesl		12,285						79,852
C2		3,900						25,350
U1	13.0 × 18.26 × 62.5		14,836			6.50	96,436	
U2	0.5 × 13.0 × 5.24 × 62.5		2,129			8.667	18,450	
F	0.5 × 12.53 × 1577		9,880			4.177	41,268	
		27,966	26,845				156,154	181,779
		V = 1,121						M = 25,625

	Sec. (3), Loading No. 5	↓	↑	→	←	Arm	↻	↺
Ww		1,625				6.50		10,563
Wesl		12,285						79,852
C2		3,900						25,350
U1	13.0 × 11.0 × 62.5		8,938			6.50	58,094	
F1	13.0 × 767		9,971			6.50	64,812	
F2	0.5 × 13.0 × 592		3,848			4.333	16,673	
		17,810	22,757				139,579	115,765
		V = 4,947					M = 23,814	

SUBJECT: Quaboag River, West Warren, Mass.
COMPUTATION: "T" Section Design
COMPUTED BY: C.C.C. CHECKED BY: R.A.K. DATE: 6-16-61

Base Steel:



1/4" = 1'-0"

1" = 1,000 □" A_s

	Top Face				Bottom Face			
Section	①	②	③	④	①	②	③	④
Loading No	#2	#2	#2	#5	#5	#5	#5	#2
t"	22.	26.	30.	30.	22.	26.	30.	30.
d"	17.5	21.5	25.5	25.5	17.5	21.5	25.5	25.5
V#	3,210.	4,820.	3,671.	1,386.	794.	2,441.	4,947.	4,145.
v	17.3	21.2	13.6	5.1	4.3	10.7	18.3	15.3
M^{K'}	6,843.	25,225.	44,594.	2,529.	1,414.	8,113.	23,814.	9,074.
Req'd. d	6.55	12.55	16.70	3.98	2.98	7.13	12.20	7.53
Req'd. A_s	0.27	0.80	1.19	0.07	0.05	0.26	0.63	0.24
Use	#8@1'-4"	←	#8@8"	#8@1'-4"	#6@1-4	→	#6@8"	#6@1-4
Actual A_s	0.59		1.19	0.59	.33		.66	.33
Σ_o	2.4		4.7	2.4	1.8		3.5	1.8
u	85.	54.	35.	26.	29.	71.	63.	104.

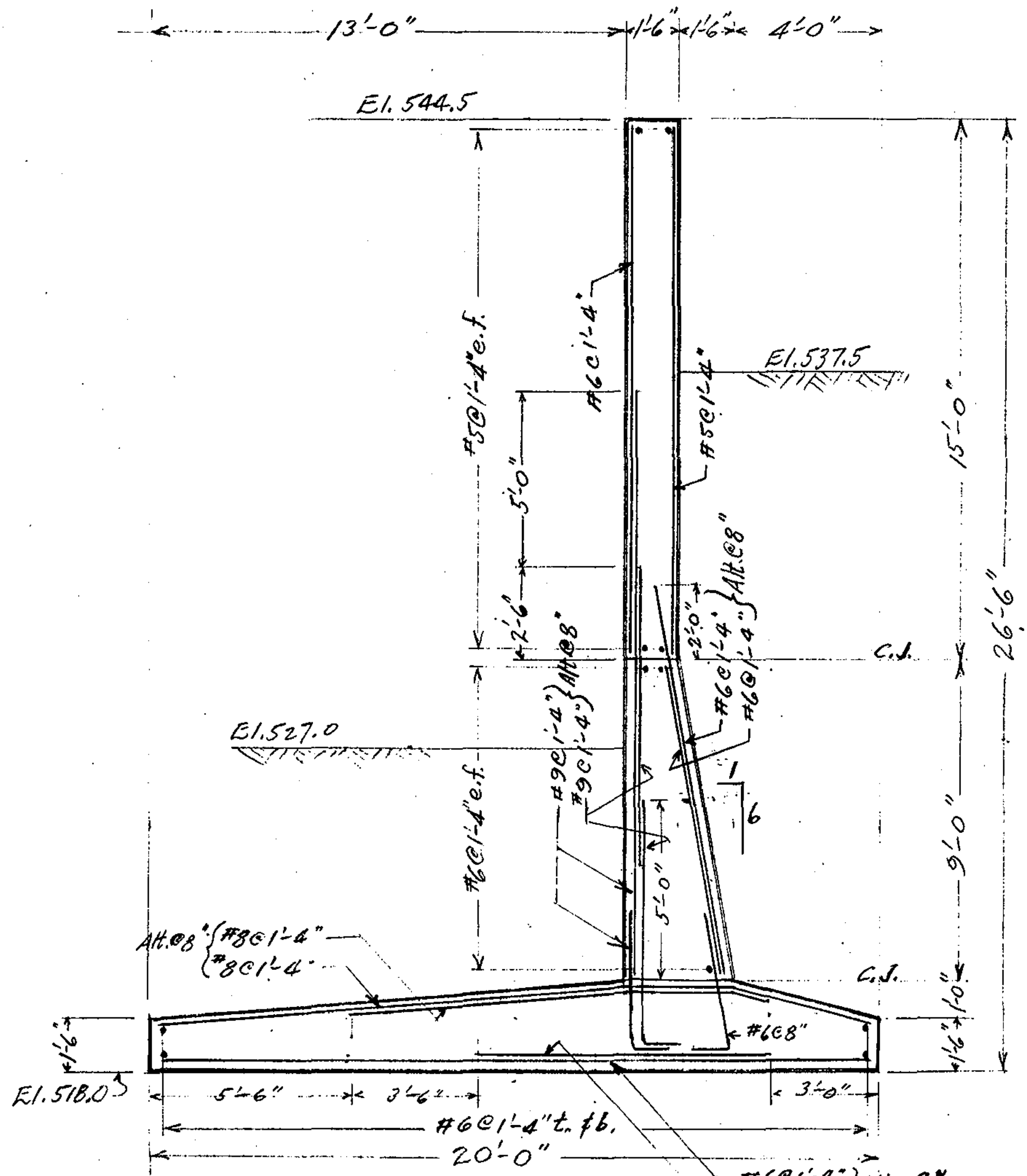
$b = 12''$
$j = .885$
$K = 160$
$a = 1.475$
$d = t - 4.5''$
$v = \frac{V}{bjd} = .09416\frac{V}{d}$

$$\text{Req'd } d = \sqrt{\frac{M^{K'}}{.160}} = 2.5\sqrt{M^{K'}}$$

$$\text{Req'd } A_s = \frac{M^{K'}}{a\,d} = .6780\frac{M^{K'}}{d}$$

$$u = \frac{V}{\Sigma_o\, j\, d}$$

SUBJECT Quaboag River, West Warren, Mass.
COMPUTATION "T" Section Design
COMPUTED BY C.C.C. CHECKED BY R.A.K. DATE 6-14-61

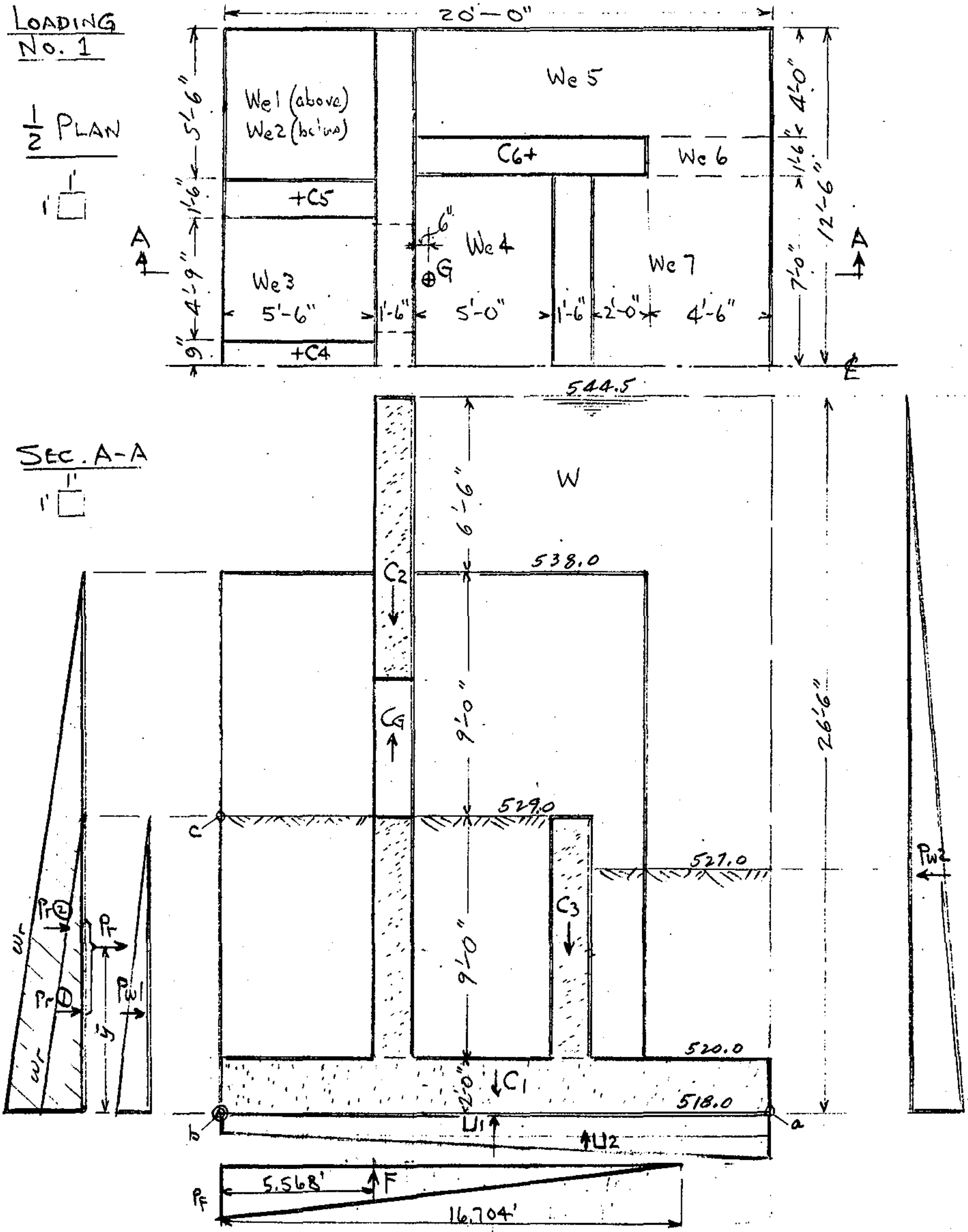


SECTION C-C
1/4" = 1'-0"

SUBJECT Quaboag River, West Warren, Mass.

COMPUTATION Gate Monolith

COMPUTED BY C.L.L. CHECKED BY R.A.K. DATE 6-19-61

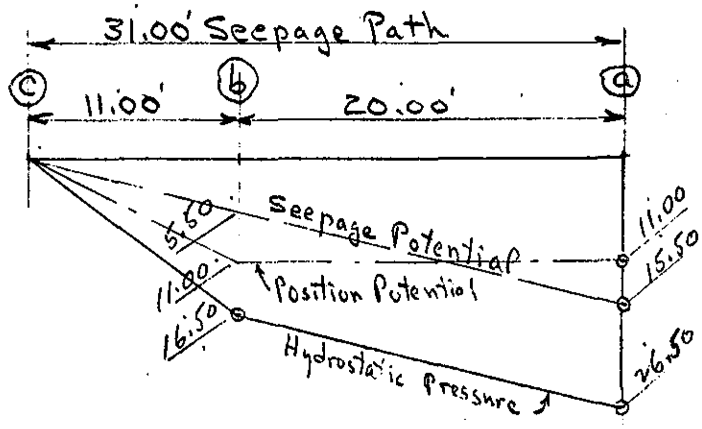


SUBJECT Quaboag River, West Warren, Mass.
COMPUTATION Gate Monolith
COMPUTED BY C.C.C. CHECKED BY R.A.K. DATE 6-19-61

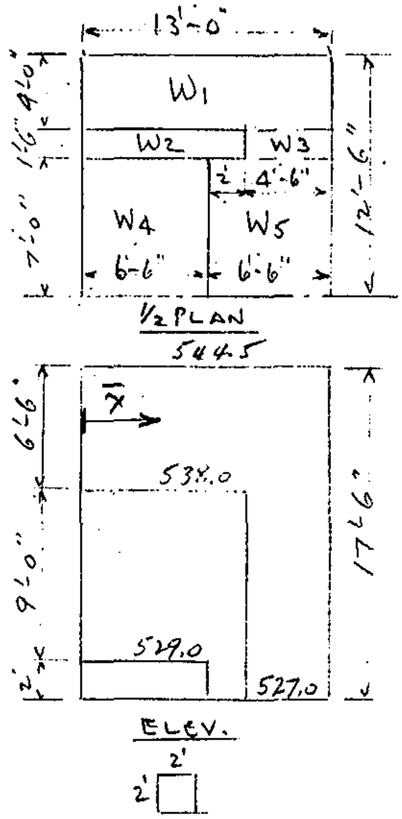
Uplift Diagram

Scales :

Horiz. : 1" = 10'
Vert. : 1" = 20' Head



Calc. of "W" :



$W_1 = 13.0 \times 4.0 \times 17.5 = 910.$ cu.ft. $\times 6.5 = 5,915.$

$W_2 = 8.5 \times 1.5 \times 6.5 = 82.9 \times 4.25 = 352.$

$W_3 = 4.5 \times 1.5 \times 17.5 = 118.1 \times 10.75 = 1,270.$

$W_4 = 6.5 \times 7.0 \times 15.5 = 705.3 \times 3.25 = 2,292.$

$W_5 = 6.5 \times 7.0 \times 17.5 = \underline{796.2} \times 9.75 = \underline{7,763.}$

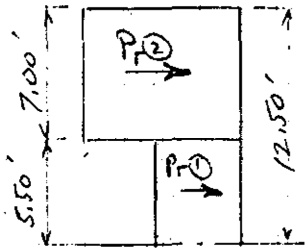
2,612.5 cu.ft. 17,592

$$\bar{x} = \frac{17,592}{2,612.5} = \underline{6.734'}$$

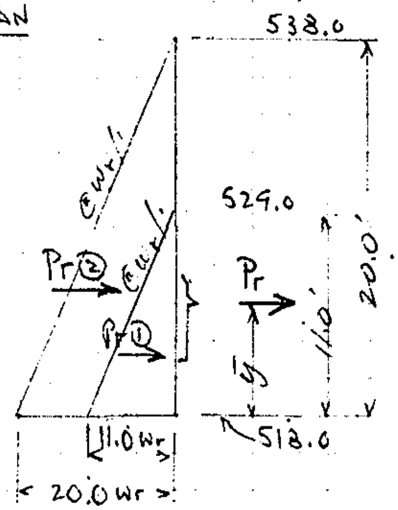
$$W = 2612.5 \times 62.5 = \underline{163,281.^{\#}}$$

SUBJECT: QUABOAG RIVER, WEST WARREN, MASS.
COMPUTATION: GATE MONOLITH
COMPUTED BY: C.C.C. CHECKED BY: R.A.K DATE: 6-19-61

Calc. of $\bar{y}$ for Pr① + Pr②:



½ PLAN



ELEV.

$$Pr① = 5.50 \times \frac{1}{2}(11.00\,w_r)\,11.00 = 332.75\,w_r \quad \times \frac{11.00}{3} = 1,220.\,w_r$$

$$Pr② = 7.00 \times \frac{1}{2}(20.00\,w_r)\,20.00 = 1,400.00\,w_r \quad \times \frac{20.00}{3} = 9,333.\,w_r$$

$$P_r = 1732.75\,w_r \qquad 10,553.\,w_r$$

$$\bar{y} = \frac{10,553\,w_r}{1,732.75\,w_r} = \underline{6.09'}$$

SUBJECT Quaboag River, West Warren, Mass.
COMPUTATION Gate Monolith
COMPUTED BY C.C.C. CHECKED BY R.A.K. DATE 6-19-61

	$\Sigma M_b = 0$	↓	↑	→	←	Arm	↷	↶
C1	20×12.5×2.0×150.	75,000				10.0	750,000.	
C2	1.5×12.5×24.5×150.	68,906				6.25	430,664.	
C3	1.5×7.0×9.0×150.	14,175				12.75	180,731.	
C4	5.5×0.75×18.0×150.	11,138				2.75	30,628.	
C5	5.5×1.5×18.0×150.	22,275				2.75	61,256.	
C6	8.5×1.5×18.0×150.	34,425				11.25	387,281.	
G	Gate=4,000+Fl.Stand=250	4,250.				7.50	31,875.	
We1	5.5×5.5×9.0×130.	35,393				2.75	97,329.	
We2	5.5×5.5×9.0×135.	36,754				2.75	101,073.	
We3	5.5×4.75×9.0×135.	31,742.				2.75	87,290.	
We4	5.0×7.0×9.0×135.	42,525				9.5	403,988.	
We5	13.0×4.0×7.0×125.	45,500.				13.5	614,250.	
We6	4.5×1.5×7.0×125.	5,906.				17.75	104,836	
We7	6.5×7.0×7.0×125.	39,813				16.75	666,859.	
W	See separate calculation	163,281				13.734	2,242,501.	
CG	1.5×4.0×5.0×150.		4,500.			6.25		28,125.
U1	20.0×12.5×16.5×62.5		257,813			10.0		2,578,125.
U2	0.5×20.0×12.5×10.0×62.5		78,125.			13.333		1,041,641
F			① 290,645			⑤ 5.568		④ 1,618,434
Pw1	0.5×12.5×11.0×16.5×62.5			70,898		3.667	259,985.	
Pr				② 203,418		6.09	③ 1,238,816.	
Pw2	0.5×12.5×26.5×26.5×62.5				274,316	8.833		2,423,037
		631,083	631,083	274,316	274,316		7,689,362.	7,689,362

$$F = \frac{1}{2} \times 16.704 \times 12.5\, p_F$$

$$p_F = \frac{2 \times 290,645}{16.704 \times 12.5} = \underline{2,784.\ psf.}$$

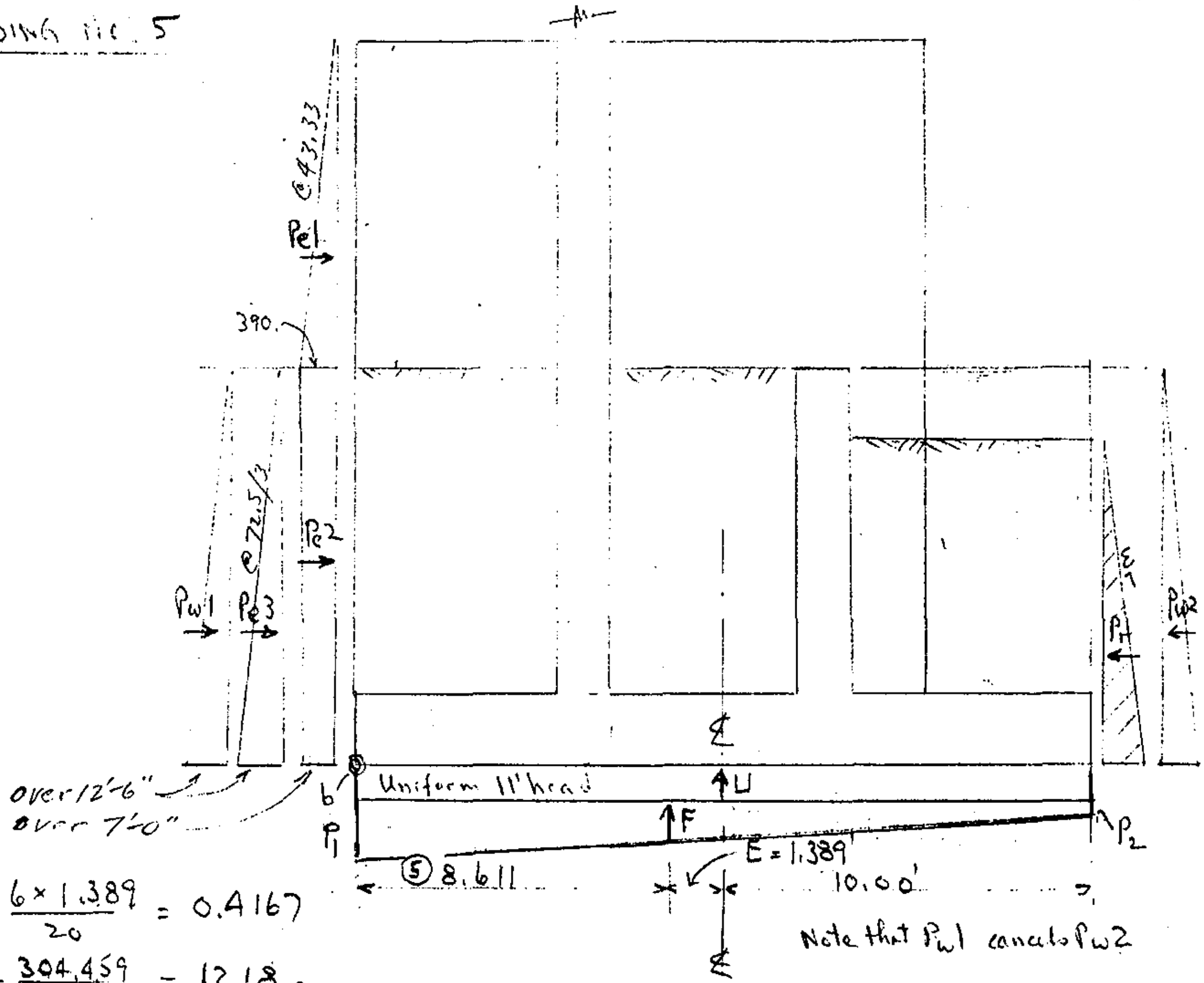
$$\omega_r = \frac{②\, P_r}{1732.75} = \underline{117.\ \#/0'}$$

SUBJECT Quaboag River, West Warren, Mass.

COMPUTATION Gate Monolith

COMPUTED BY C.C.C. CHECKED BY DATE 6-20-61

LOADING NO. 5



$$\frac{6E}{L} = \frac{6 \times 1.389}{20} = 0.4167$$

$$\frac{F}{BL} = \frac{304,459}{12.5 \times 20} = 1218.$$

$$P_1, P_2 = \frac{F}{BL}\left(1 \pm \frac{6E}{L}\right) = 1,725.\ psf;\quad 710.\ psf$$

$$w_r = \frac{P_r}{0.5 \times 12.5 \cancel{\times} (9.0)^2} = \frac{60,603}{506.25} = 120.\ psf./1$$

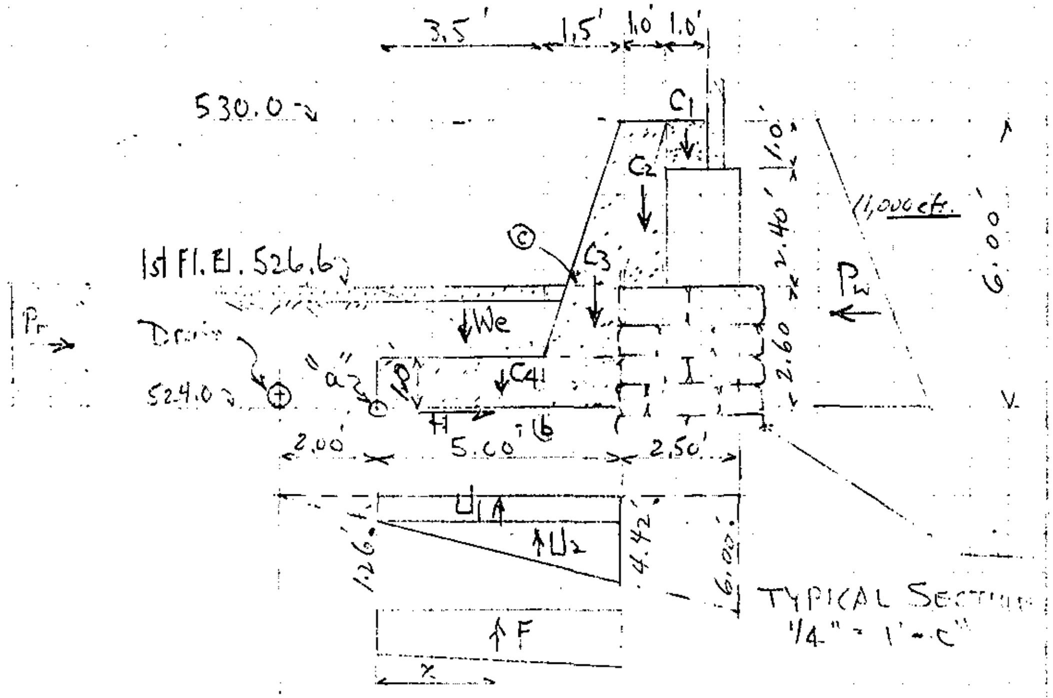
$\Sigma M_b = 0$		↓	↑	→	←	ARM	↻	↺
C1-6, G, We1-7		467,802					3,948,060.	
W1	13.0 × 4.0 × 2.0 × 62.5	6,500				13.5	87,750	
W3	4.5 × 1.5 × 2.0 × 62.5	844				17.75	14,977.	
W5	6.5 × 7.0 × 2.0 × 62.5	5,688.				16.75	95,266.	
CG			4,500.					28,125.
U	20.0 × 12.5 × 11.0 × 62.5		171,875			10.0		1,718,750.
F			① 304,459			⑤ 8.611		④ 2,621,584
Pe1	0.5 × 7.0 × 9.0 × 390.			12,285		14.0	171,990.	
Pe2	7.0 × 11.0 × 390.			30,030		5.5	165,165.	
Pe3	0.5 × 12.5 × 11.0 × 266.			18,288		3.667	67,060.	
Pr					② 60,603	3.0		③ 181,809
		480,834	480,834	60,603	60,603		4,550,268	4,550,268.

SUBJECT: QUABOAG RIVER, WEST WARREN, MASS.

COMPUTATION: BUILDING NO. 7

COMPUTED BY: C.C.C. CHECKED BY: GFH DATE: 3-27-61

LOADING NO. 1



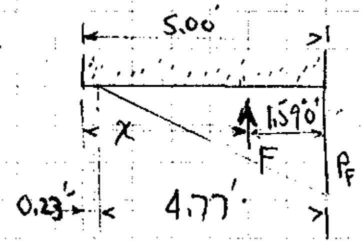
	$\Sigma M_a = 0$	↓	↑	→	←	ARM	↻	↺
C1	1.0 × 1.0 × 150	150				6.50	975	
C2	1.0 × 3.4 × 150	510				5.50	2,805	
C3	½ 1.5 × 5.0 × 150	563				4.50	2,531	
C4	5.0 × 1.0 × 150	750				2.50	1,875	
We	3.5 × 1.6 × 115	644				1.75	1,127	
U1	5.0 × 1.26 × 62.5		394			2.50		984
U2	½ × 5.0 × 3.16 × 62.5		494			3.333		1646
F			① 1729		② x =	2.564	③	4,433
H				1125		0.0	0	
Pw	½ (6.0)² 62.5				1125	2.00		2,250
		2,617	2,617	1,125	1,125		9,313	9,313

Sliding: H/F = 1125/1729 = 0.65 Too high. Max 0.4. Assume sliding, Add Pr. to ftg.

SUBJECT: QUABOAG RIVER, WEST WARREN, MASS.
COMPUTATION: BUILDING No. 7
COMPUTED BY: C.C.C. CHECKED BY: GFH DATE: 3-27-60

	$\Sigma M_a = 0$	↓	↑	→	←	Arm	↷	↶
C_1		150					975	
C_2		510					2805	
C_3		563					2531	
C_4		750					1875	
We		644					1127	
U_1			394					984
U_2			494					1646
F			① 1729			⑤ $x =$	3.410	④ 5896
P_r				② 1125		1.30	③ 1463	
P_w					1125			2,250
		2,617	2,617	1125	1125		10,776	

$P_F = 2 \times 1729 / 4.77 = 725$ psf



Structural Design:

Sec © : $M_c < \frac{1}{6}(3.4)^3\, 62.5 < 409^{\#'}$ $\quad d = \sqrt{409/160} = 1.6''$

	Sec ⓑ : ΣM_b	↓	↑	Arm	↷	↶
C_4	$3.5 \times 1.0 \times 150$	525		1.75	919	
We	$3.5 \times 1.6 \times 115$	644		1.75	1127	
U_1	$3.5 \times 1.26 \times 62.5$		276	1.75		482
U_2	$0.5 \times 3.5 \times 2.21 \times 62.5$		242	2.33		563
F	$0.5 \times 3.27 \times 497$		813	1.09		886
		1169	1331		2046	1931
			162		115	

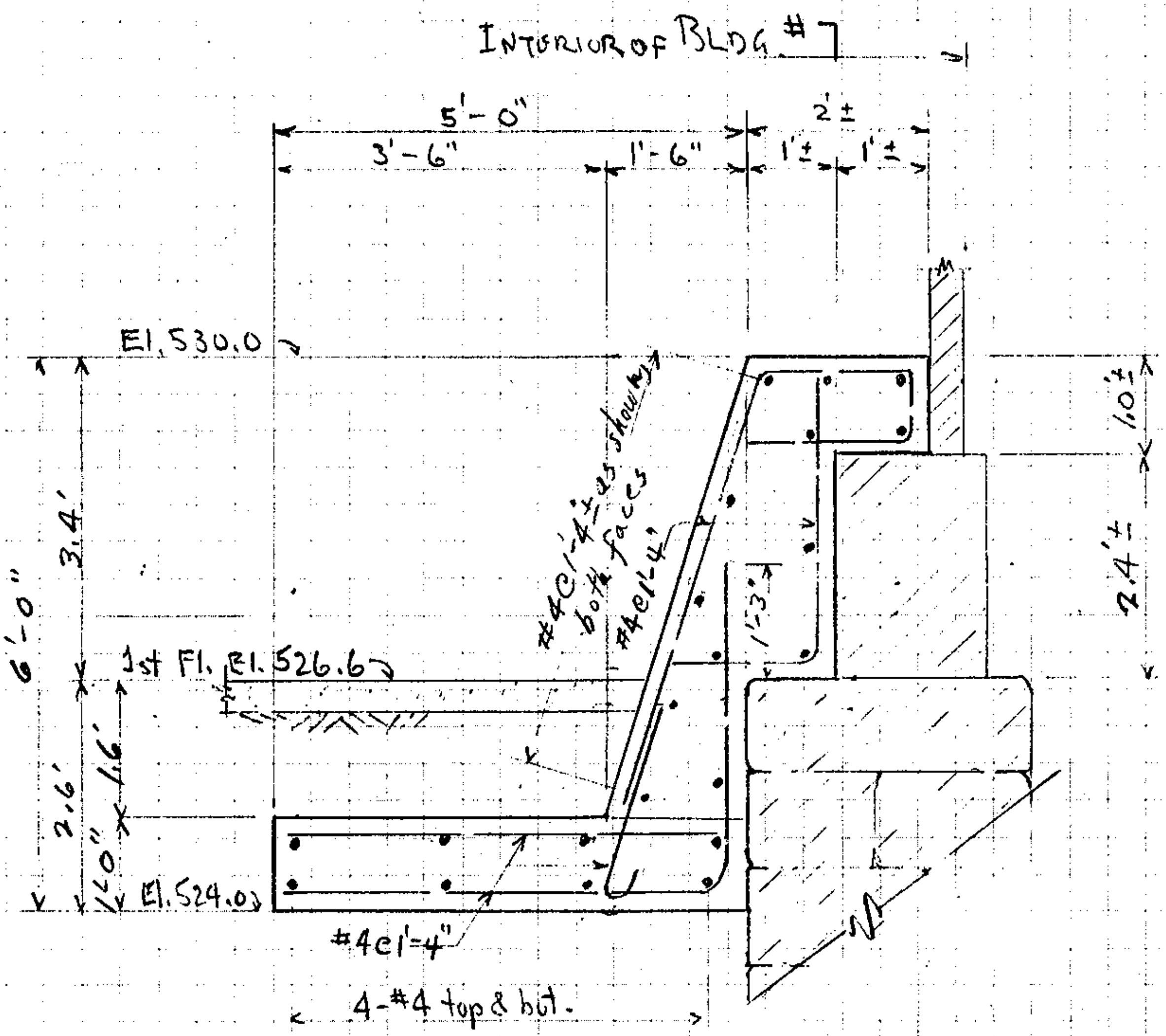
$d = \sqrt{115/160} = < 1''$

Use min. steel throughout.

Slab: $.001\,bt = .001 \times 12 \times 12 = 0.144\ \square''/'$ each face.

Use #4 @ 16" both faces $= 0.15\ \square''/'$

SUBJECT Quaboag River, West Warren, Mass.
COMPUTATION Building No. 7
COMPUTED BY C.C.C. CHECKED BY GFH DATE 3-27-60



TYPICAL SECTION

1/2" = 1'-0"

TEST DATA SUMMARY

Sheet 1 of 1

FEATURE W. WARREN, MASS. LOCAL PROTECTION

BORING NO.	SAMPLE NO.	DEPTH OR ELEV OF SAMPLE	LABORATORY CLASSIFICATION	MECHANICAL ANALYSIS GRAVEL %	MECHANICAL ANALYSIS SAND %	MECHANICAL ANALYSIS FINES %	MECHANICAL ANALYSIS D_{10}	ATTERBERG LIMITS LL	ATTERBERG LIMITS PL	SPECIFIC GRAVITY G	NATURAL WATER CONTENT %	NATURAL DRY DENSITY LBS/CU FT	COMPACTION DATA OPTIMUM WATER %	COMPACTION DATA MAXIMUM DRY DENSITY LBS/CU FT	SHEAR DATA INITIAL e	SHEAR DATA DRY DENSITY LBS/CU FT	SHEAR DATA W_i %	SHEAR DATA W_f %	SHEAR DATA S_i %	SHEAR DATA TYPE TEST	SHEAR DATA SPECIMEN SIZE INCHES	SHEAR DATA TEST	SHEAR DATA σ_{III} T/SQ FT	SHEAR DATA σ_I T/SQ FT	SHEAR DATA C T/SQ FT	SHEAR DATA Ø DEGREES	PERMEABILITY e	PERMEABILITY K FT/MIN	CONSOLIDATION DATA P_o T/SQ FT	CONSOLIDATION DATA P_c T/SQ FT	CONSOLIDATION DATA C_c	CONSOLIDATION DATA t_{50}*	REMARKS
FD-1	J2	0.6-3.2	SM	25.3	51.1	23.6	.018																										
"	J6	5.8-10.0	SM	18.6	58.2	23.2	.018																										
"	J9	12.0-14.1	SM	28.2	41.8	31.0	.006																										
FD-2	J4	5.0-10.0	SP-SM	25.9	63.8	10.3	.077																										
"	J6	15.0-20.0	SM	19.0	48.5	32.5	.006	19	16																								
FD-3	J4	5.0-9.1	GP-GM	59.0	33.0	8.0	.12																										
"	J5	9.1-10.0	SM	25.7	56.6	17.7	-																										
FD-4	J3	3.5-5.0	SM	13.3	65.6	21.1	-																										
"	J5	6.3-7.6	SM	2.2	66.8	31.0	.017																										
"	J7	10.0-13.3	GM	40.5	37.5	22.0	—																										
FD-5	J2	1.8-5.0	SM	34.7	49.1	16.2	-																										
"	J6	7.6-9.5	ML	18.0	31.7	50.3	-																										

* Values at Pressure T/SQ FT

- Triaxial Compression
UC - Unconfined Compression

DS - Direct Shear
UU - Unconsolidated Undrained

- Consolidated Drained
CU - Consolidated Undrained

ENG FORM 1 DEC 51 2086

APPENDIX B

FLOOD LOSSES AND BENEFITS

PLATES

APPENDIX B

FLOOD LOSSES AND BENEFITS

1. DAMAGE SURVEYS

A damage survey was made in the flood area immediately after the 1955 flood and supplemented by a review in the Spring of 1959. These surveys consisted of detailed inspections of the industrial properties to ascertain the physical condition of the properties, the nature and amount of damages, depths of flooding, high water references and relationships between the 1955 flood and other flood stages. Damage data in the experienced flood furnished by owners and tenants were utilized, when in the judgment of the investigators the estimates were realistic. In other cases, owners' estimates were modified by investigators. Additional information was obtained from local and State officials.

Sufficient data were obtained to derive loss estimates for (1) the 1955 flood crest, (2) a stage 3 feet above the 1955 crest, and (3) intermediate stages below 1955 crest where marked changes in damage occur. The stage at which damage begins, referenced to the 1955 flood, was also determined.

2. LOSS CLASSIFICATION

Losses in the project area were principally industrial, with some loss to railroad and highway.

Primary losses have been classified as physical and non-physical. Physical losses comprise primary losses, such as damage to structures, machinery, and inventories, and the cost of cleanup and repairs. Non-physical losses include unrecoverable loss of business, wages or production, increased cost of operation and cost of temporary facilities.

Physical damage and a large part of the related non-physical losses were determined by direct inspection of property and evaluation of losses by property owners, tenants and field investigators.

3. RECURRING AND PREVENTABLE LOSSES

There has been no change in use in the flood area since the flood of 1955, with the exception of the one building adjacent to the river which has changed tenants. Estimates have been made of the recurring damages that would be experienced with various flood stages above and below the 1955 flood level. A recurrence of the 1955 stages would cause damages estimated at $1.1 million at 1961 price levels.

4. AVERAGE ANNUAL LOSSES

Estimated recurring losses were converted to average annual losses as a basis for determining annual benefits for use in economic evaluation. Estimated annual losses were derived by correlation of stage-damage, stage-discharge, and discharge frequency curves, to produce damage-frequency relationships in accordance with standard Corps of Engineers practice. Annual losses were derived only for those floods having a frequency of more than once in 400 years, inasmuch as the relative accuracy of discharge frequency, and stage-frequency relationships deteriorates sharply for the rarer floods. Annual losses for the flood area are estimated at $18,700 at 1961 prices.

Plates E-1 through E-7 show procedures used in converting recurring stage-damage data to curves of damage-frequency.

5. ANNUAL BENEFITS

a. Flood damage prevention benefits. Average annual flood damage prevention benefits were determined by deriving the difference between average annual losses under existing conditions and those losses remaining after construction of protective works. Annual losses resulting from floods having a frequency of more than once every 300 years will be eliminated by construction of a dike in the headwater portion of the project, and by improvement of the channel downstream. Average annual flood damage prevention benefits accruing to the protective works amount to $17,400.

b. Other tangible benefits. No increase in utilization of lands and buildings is expected to follow construction of the local protection project. Land available in the flood-free areas in the project vicinity have shown little development in the past few years. Few structural changes have been noted in this locality in more recent times. Construction of the project would relieve the threat of flooding and encourage the present industrial concerns to remain in the district.

c. Intangible benefits. Significant intangible benefits will be realized from construction of the proposed local protection project. The safety of the entire village of West Warren against fire is dependent upon the pumping facilities located in the project area. The assurance of non-interruption of employment for nearly 1,000 people will have a substantial effect on the economy of West Warren and surrounding communities. In addition, construction of the project would greatly lessen the flood threat to life and the potential danger of disease inherent in all floods.

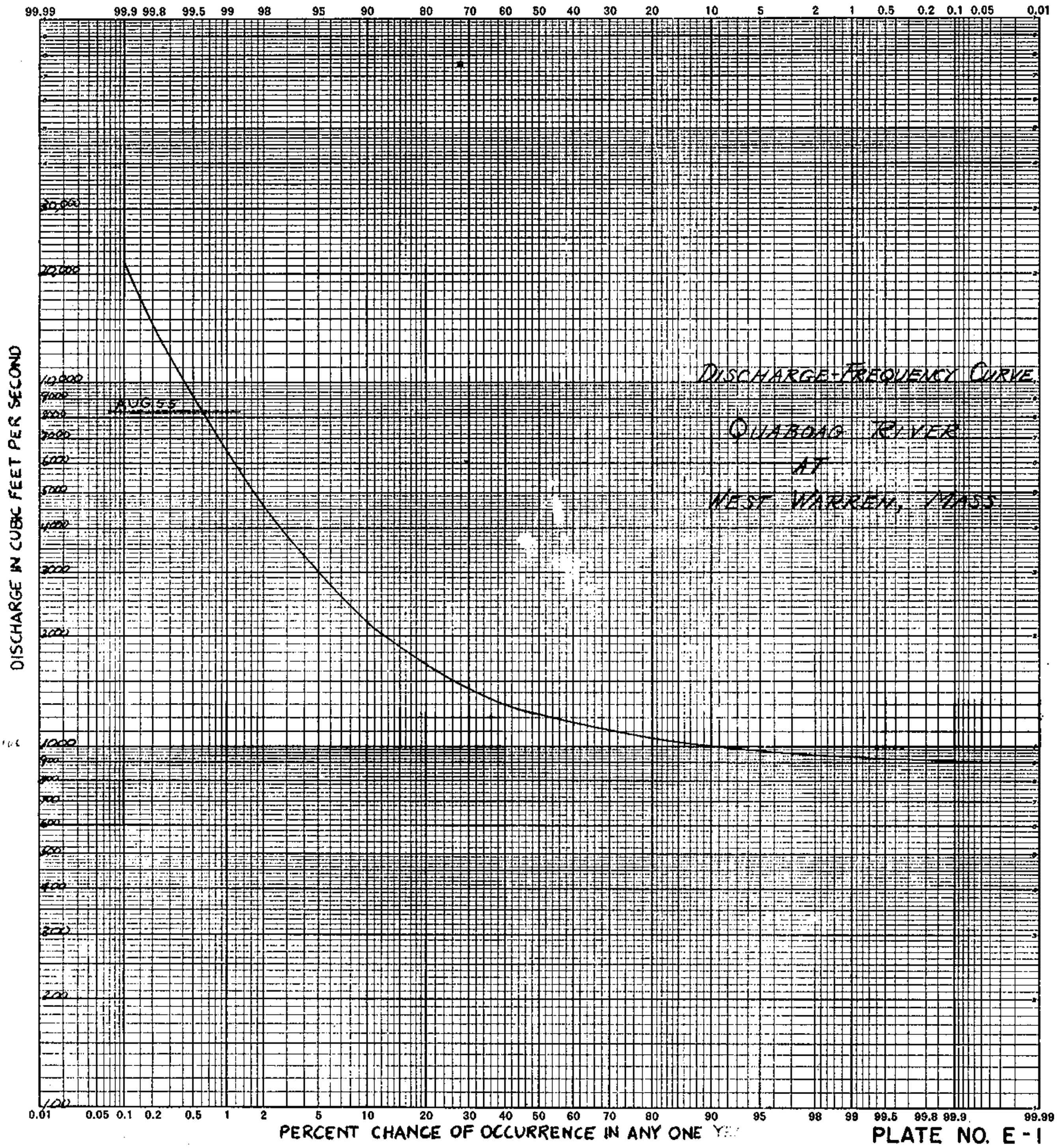

DISCHARGE-FREQUENCY CURVE
QUABOAG RIVER
AT
WEST WARREN, MASS
AUG 55
DISCHARGE IN CUBIC FEET PER SECOND
PERCENT CHANCE OF OCCURRENCE IN ANY ONE
99.99 99.9 99.8 99.5 99 98 95 90 80 70 60 50 40 30 20 10 5 2 1 0.5 0.2 0.1 0.05 0.01
0.01 0.05 0.1 0.2 0.5 1 2 5 10 20 30 40 50 60 70 80 90 95 98 99 99.5 99.8 99.9 99.99
30,000
20,000
10,000
9000
8000
7000
6000
5000
4000
3000
2000
1000
900
800
700
600
500
400
300
200
100

PLATE NO. E-1

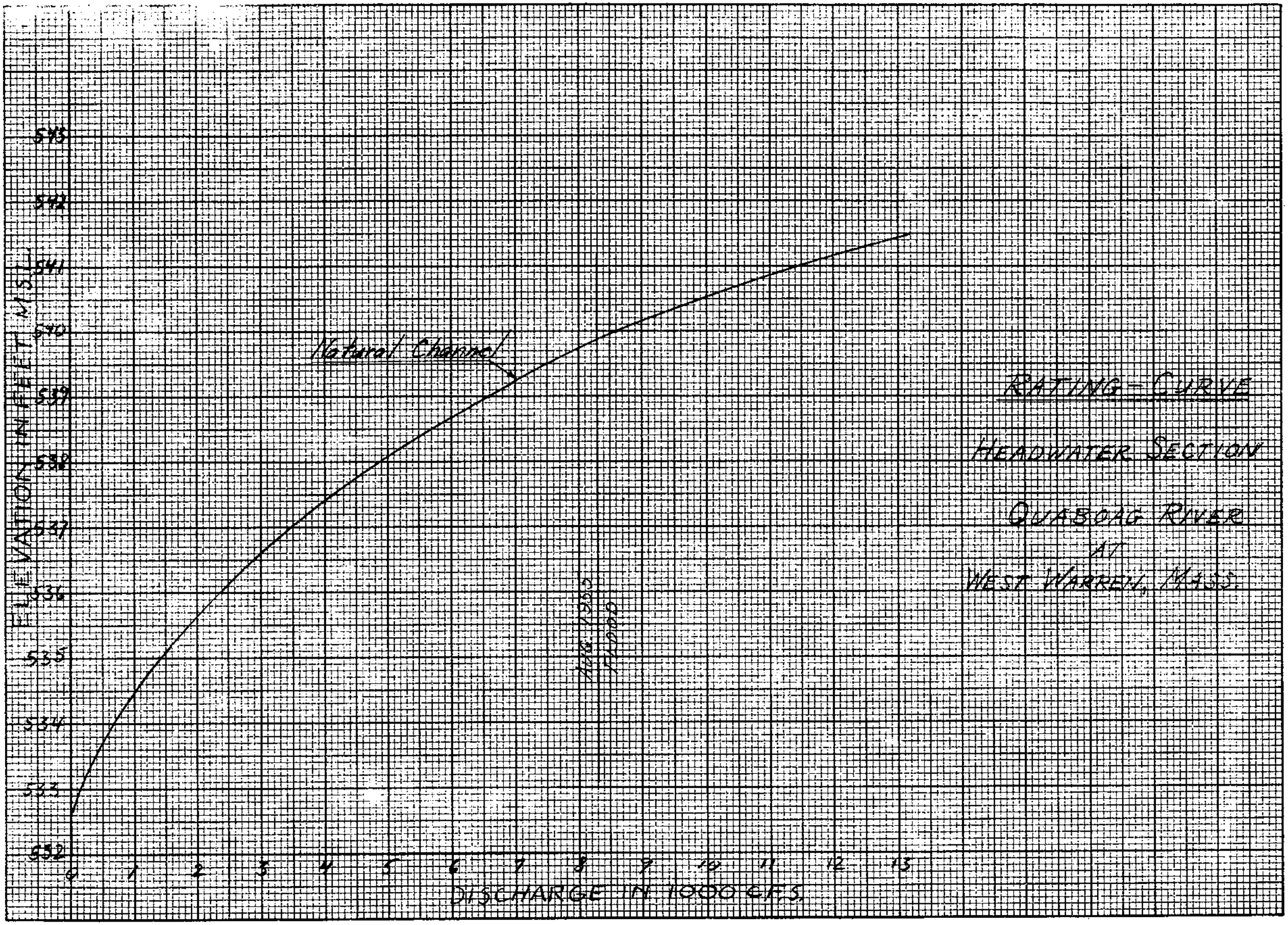
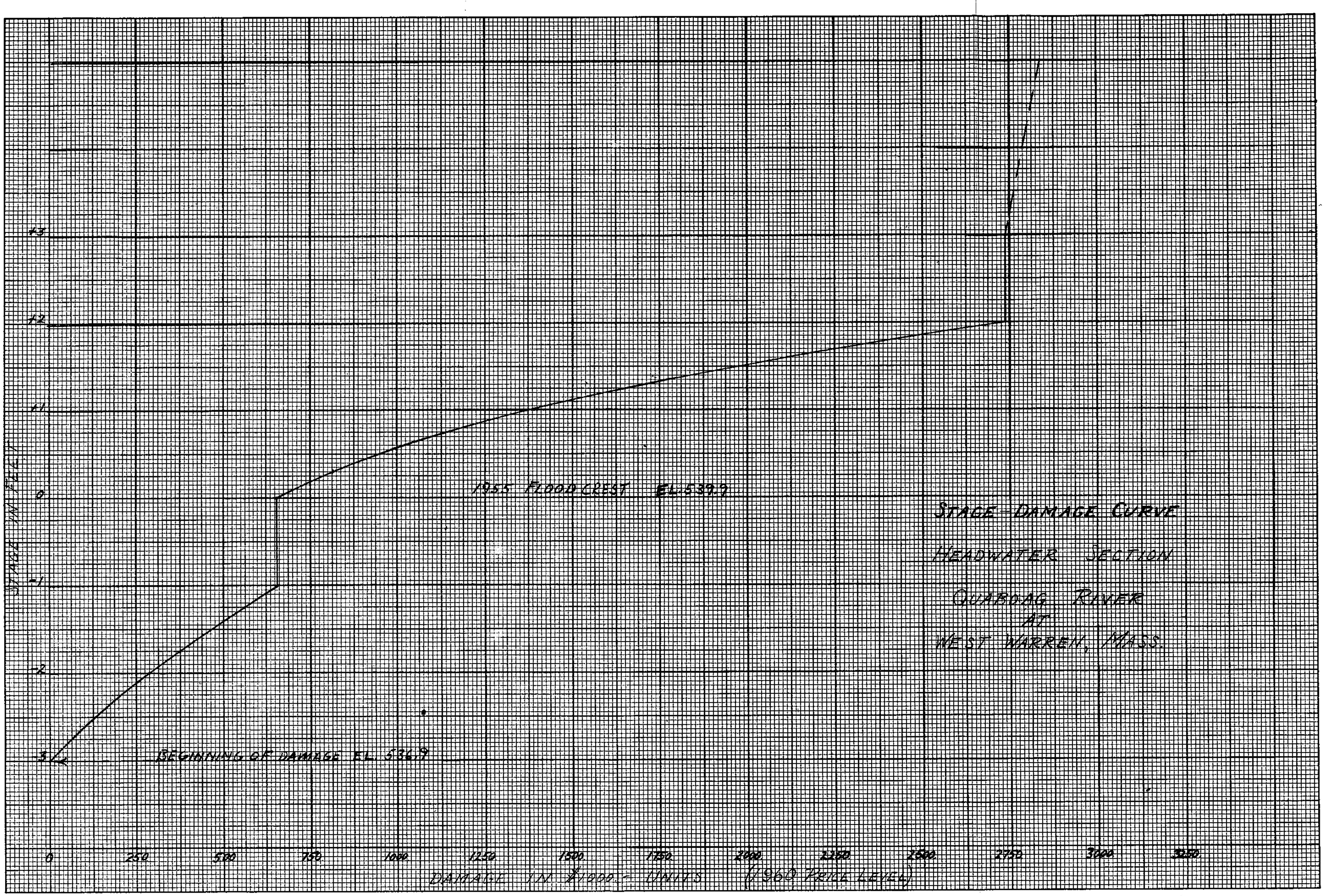
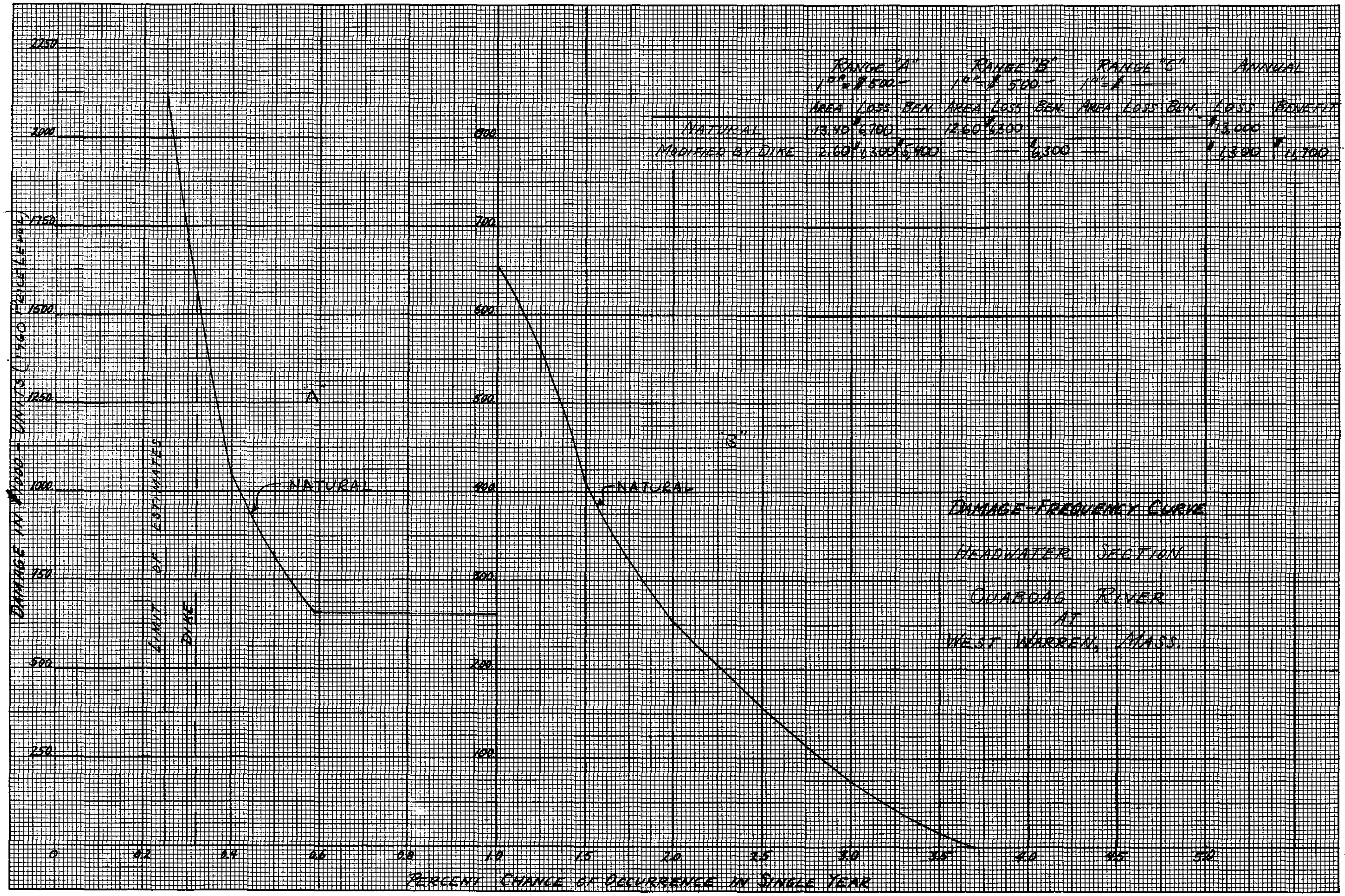

RATING CURVE
HEADWATER SECTION
QUABOAG RIVER
AT
WEST WARREN, MASS.
Natural Channel
AUG. 1955 FLOOD
ELEVATION IN FEET M.S.L.
DISCHARGE IN 1000 C.F.S.

PLATE NO. E-2



STAGE IN FEET
+3
+2
+1
0
-1
-2
-3
1955 FLOOD CREST EL. 539.9
BEGINNING OF DAMAGE EL. 536.9
0
250
500
750
1000
1250
1500
1750
2000
2250
2500
2750
3000
3250
DAMAGE IN $1000 - UNITS (1960 PRICE LEVEL)
STAGE - DAMAGE CURVE
HEADWATER SECTION
QUABOAG RIVER
AT
WEST WARREN, MASS.

PLATE NO. E-3



	RANGE "A" 1"² = $500.–			RANGE "B" 1"² = $500.–			RANGE "C" 1"² = $ ——			ANNUAL	
	AREA	LOSS	BEN.	AREA	LOSS	BEN.	AREA	LOSS	BEN.	LOSS	BENEFIT
NATURAL	13.40	$6,700	—	12.60	$6,300	—	—	—	—	$13,000	—
MODIFIED BY DIKE	2.60	$1,300	$5,400	—	—	$6,300	—			$1,300	$11,700

PLATE NO. E-4

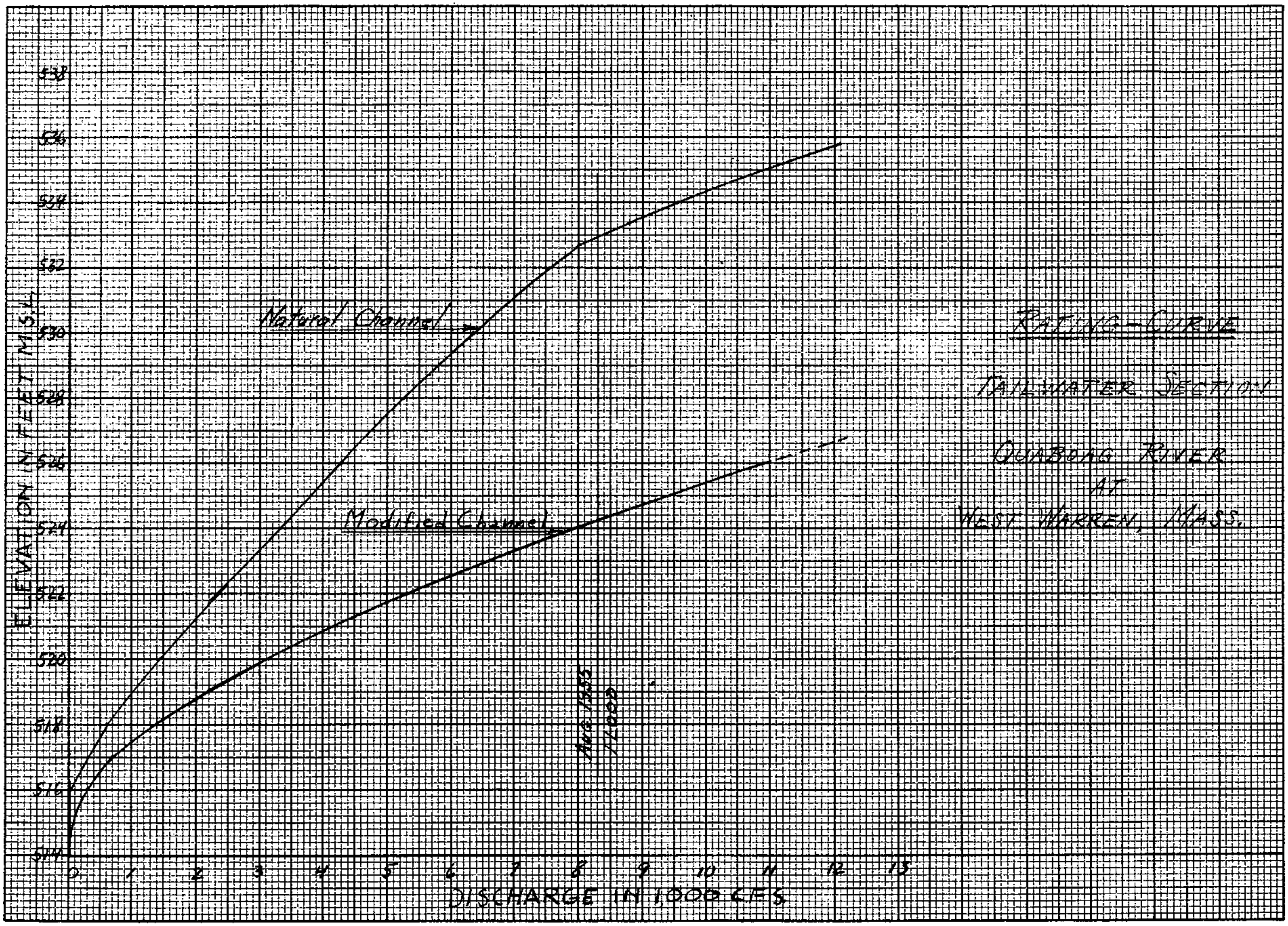
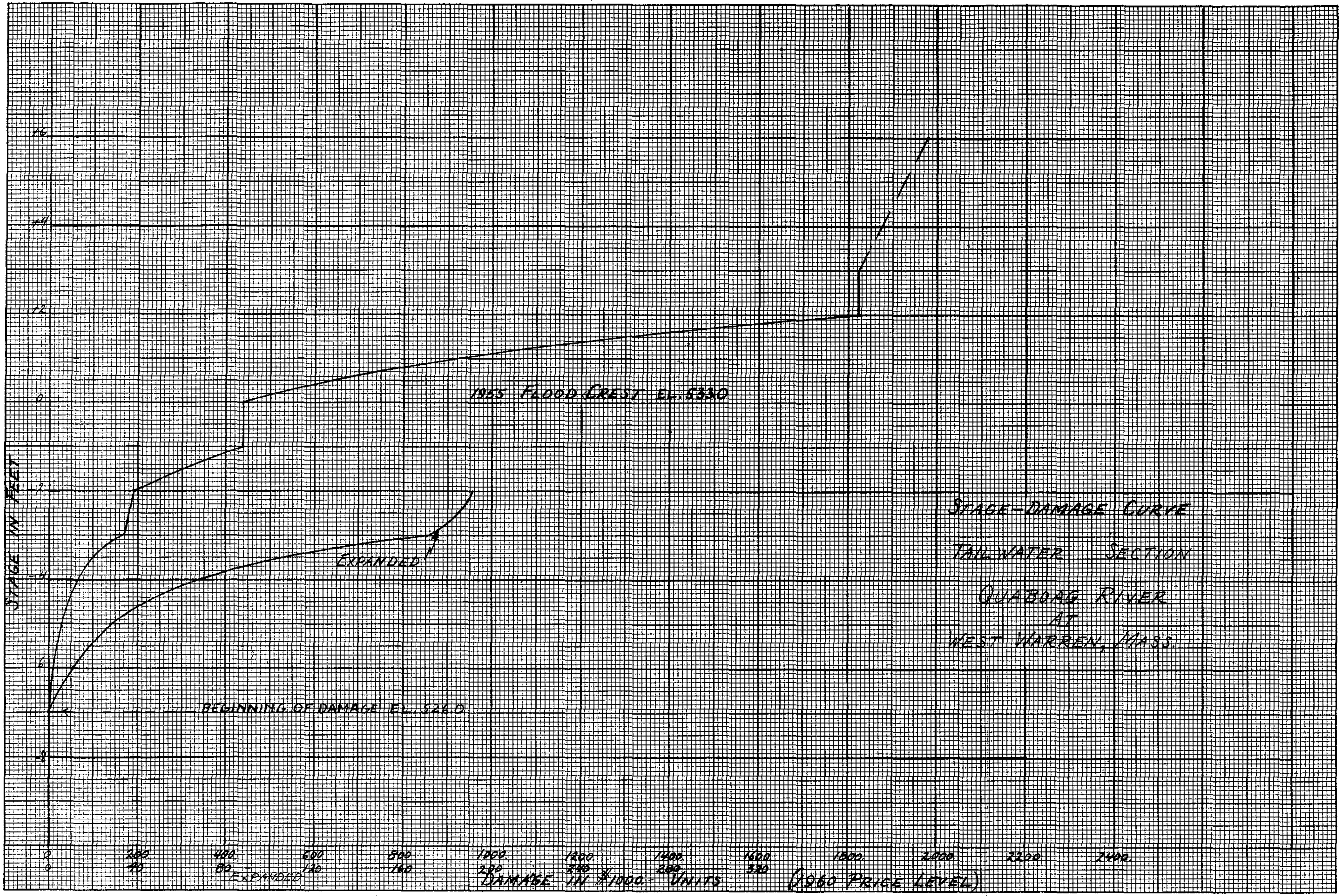
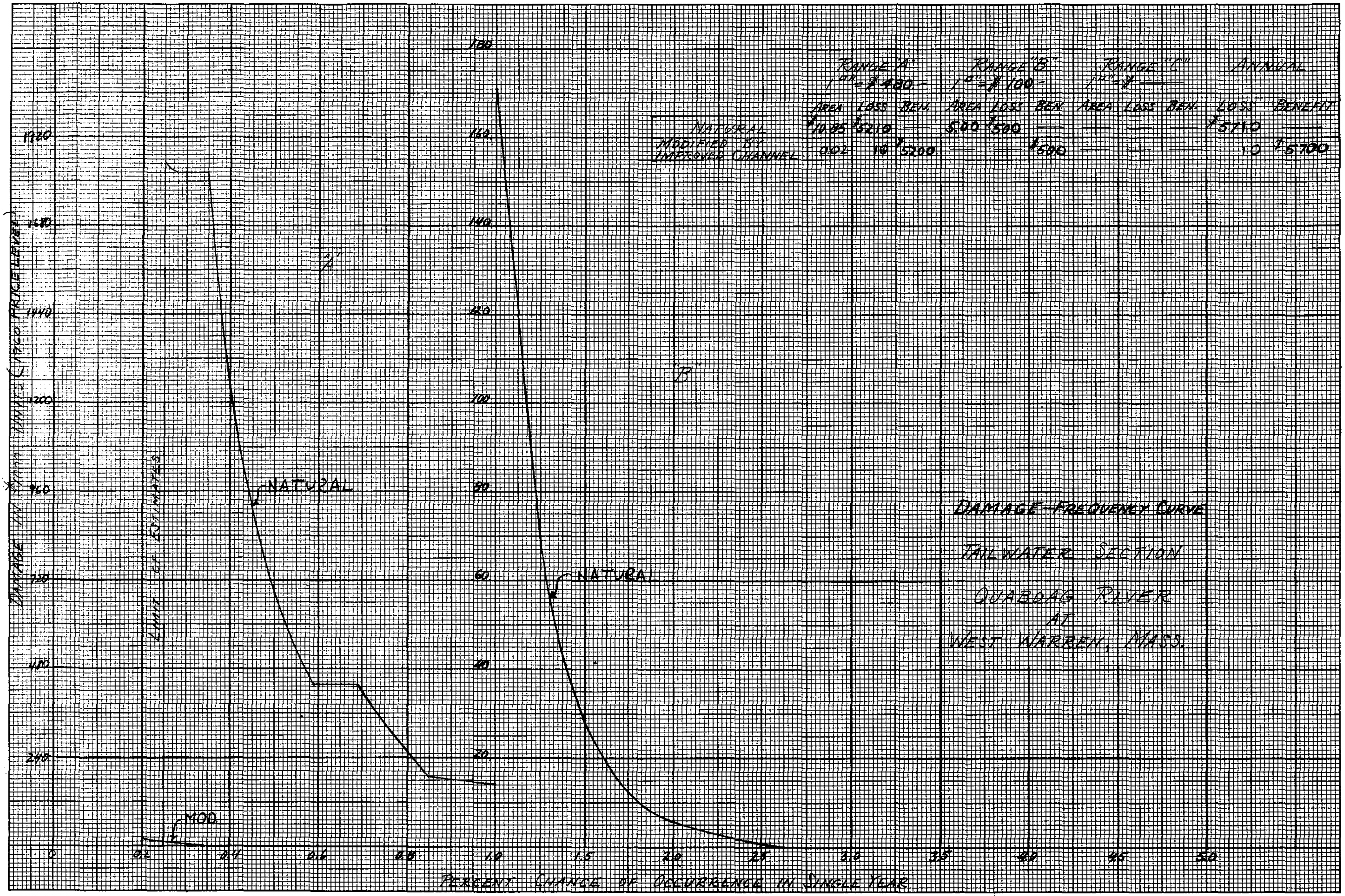

RATING - CURVE
TAILWATER SECTION
QUABOAG RIVER
AT
WEST WARREN, MASS.
ELEVATION IN FEET M.S.L.
538
536
534
532
530
528
526
524
522
520
518
516
514
Natural Channel
Modified Channel
Aug 1955
0
1
2
3
4
5
6
7
8
9
10
11
12
13
DISCHARGE IN 1000 CFS

PLATE NO. E-5



STAGE IN FEET
16
14
12
10
8
6
4
2
0
1955 FLOOD CREST EL. 533.0
EXPANDED
BEGINNING OF DAMAGE EL. 526.0
STAGE-DAMAGE CURVE
TAILWATER SECTION
QUABOAG RIVER
AT
WEST WARREN, MASS.
0 200 400 600 800 1000 1200 1400 1600 1800 2000 2200 2400
0 40 80 120 160 200 240 280 320
EXPANDED
DAMAGE IN $1000 - UNITS (1960 PRICE LEVEL)

PLATE NO. E-6



	RANGE "A" 1" = $480.-			RANGE "B" 1" = $100.-			RANGE "C" 1" = $ —			ANNUAL	
	AREA	LOSS	BEN.	AREA	LOSS	BEN.	AREA	LOSS	BEN.	LOSS	BENEFIT
NATURAL	$10.85	$5210	—	5.00	$500	—	—	—	—	$5710	—
MODIFIED BY IMPROVED CHANNEL	0.02	10	$5200	—	—	$500	—	—	—	10	$5700

PLATE NO. E-7

APPENDIX C

LETTERS OF CONCURRENCE AND COMMENT

Exhibit No.	Agency	Letter Dated
1	Town of Warren, Massachusetts	June 6, 1961
2	The Commonwealth of Massachusetts	July 19, 1961
3	U. S. Department of Agriculture Soil Conservation Service	June 14, 1961
4	U. S. Department of the Interior Fish and Wildlife Service	July 7, 1961
5	New York Central System	July 11, 1961

TOWN OF WARREN
BOARD OF SELECTMEN
TOWN HALL

June 6, 1961

Steven A. Korzec, Chairman
Alice L. Leach, Clerk
Robert W. Williams

Karl F. Eklund,
Colonel, Corps of Engineers,
Deputy Division Engineer,
U.S. Army Engineer Div., N.E.
424 Trapelo Road, Waltham 54, Mass.

Dear Colonel Eklund:

Thank you for your communication of 26 May 1961. Reference is hereby made to recent meetings with U.S. Army Engineer Division, representatives of West Warren Industries, New York Central Railroad, and our Board regarding P.L. 685 - Local Flood Protection Project, Quaboag River, West Warren Massachusetts.

Regarding proposed plans and construction, preliminary plans being on file in our office. As per your request, it is the opinion of the present Board of Selectmen, Town of Warren, that local interests are willing and able to comply with the requirements specified under the authority of P.L. 685, 84th Congress, adopted 11 July 1956.

This is not a legal commitment at this time, but we wish to state our willingness to extend our utmost cooperation.

Thanking you, we remain

Sincerely yours,
Steven A. Korzec
Board of Selectmen,
Steven A. Korzec Chairman.

SAK-R

EXHIBIT NO.1

THE COMMONWEALTH OF MASSACHUSETTS
EXECUTIVE DEPARTMENT
STATE HOUSE, BOSTON

JOHN A. VOLPE
GOVERNOR

July 19, 1961

Brigadier General Seymour A. Potter, Jr.
Division Engineer - New England Division
U. S. Corps of Engineers
424 Trapelo Road
Waltham, Massachusetts

Dear General Potter:

I am in receipt of your letter of June 9, 1961, in regard to the proposed project to provide local flood protection works along the Quaboag River in the West Warren section of the Town of Warren, Massachusetts. This project is to be constructed under Public Law 685-84th Congress, adopted in July 1956.

In your letter you point out:

1. That the estimated Federal cost of the project is $280,000.

2. That the non-Federal cost for acquiring land and for providing funds for features of the project other than flood control will be $45,000 and that the Town of Warren has given its assurance that it will contribute that sum.

The question that is raised in your letter is whether the Commonwealth of Massachusetts will assume full responsibility for all project costs in excess of the Federal cost limitation of $400,000 in the event that the cost of the final project should exceed that amount. The Commonwealth of Massachusetts has enacted

EXHIBIT NO 2

a law, Chapter 763 of the Acts of 1957, including the appropriation of the necessary funds to pay the excess cost of flood control projects constructed under this Federal flood control program.

The sums appropriated in Chapter 763, however, revert to the General Fund on July 1, 1962. As it will be impossible to know before that date whether there will be any excess in connection with the West Warren project, I am suggesting to the Division of Waterways in the Massachusetts Department of Public Works that it request that the present appropriation be continued. We consider this project an essential one for providing flood protection in the Town of Warren and will gladly assume the responsibility for any cost of construction in excess of $400,000, providing the continuation of the existing appropriation is authorized by the Legislature when it reconvenes in January.

I trust that the Secretary of the Army will authorize the allotment of the necessary appropriation for this project.

Sincerely,

John A. Volpe

Governor

UNITED STATES DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

29 Cottage Street, Amherst, Massachusetts

June 14, 1961

General Seymour A. Potter, Jr.
Division Engineer, New England
Corps of Engineers
424 Trapelo Road
Waltham 54, Massachusetts

Dear General Potter:

We have reviewed the proposed plan of protection for a local flood protection project along the Quaboag River at West Warren, Massachusetts.

We believe that this local protection project as described in your letter of June 9, 1961, is needed as part of the over-all plan for the Upper Quaboag River Watershed. As you know, the flood prevention and watershed protection work plan for the Upper Quaboag River Watershed under P. L. 566, is based on the premise that the local protection project at West Warren will be installed.

Sincerely yours,

Benj. Isgur

Benjamin Isgur,
State Conservationist.

EXHIBIT NO. 3

UNITED STATES
DEPARTMENT OF THE INTERIOR
FISH AND WILDLIFE SERVICE
BUREAU OF SPORT FISHERIES AND WILDLIFE
59 TEMPLE PLACE
BOSTON, MASSACHUSETTS

ADDRESS ONLY THE REGIONAL DIRECTOR

NORTHEAST REGION
(REGION 5)
MAINE
NEW HAMPSHIRE
NEW YORK
VERMONT
PENNSYLVANIA
MASSACHUSETTS
NEW JERSEY
RHODE ISLAND
DELAWARE
CONNECTICUT
WEST VIRGINIA

July 7, 1961

Division Engineer
New England Division
U. S. Army Corps of Engineers
424 Trapelo Road
Waltham 54, Massachusetts

Dear Sir:

Reference is made to your letter of June 9, 1961 regarding a proposed local protection project along the Quaboag River at West Warren, Massachusetts, and in which you request comments from this Bureau regarding the effects of the project on the fish and wildlife resources. This letter constitutes our conservation and development report and has the concurrence of the Massachusetts Division of Fisheries and Game.

It is our understanding that the proposed plan of protection includes a dike and flood wall upstream of the existing dam; the construction of concrete buttress walls to protect Buildings No. 7 and 11; the removal of the existing stone arch bridge; the removal and replacement of an oil pipeline bridge by a new bridge with adequate waterway; channel excavation in the Quaboag River below the existing dam; rock slope protection along sides and bottom of the river; and the reconstruction of the north pier footing of the overhead South Street Bridge to permit channel deepening at this location.

We have considered the impact of this project and conclude that it will have no significant effect on fish or wildlife resources.

We would appreciate being advised if any major changes are made in the project plan, in order that a new report may be prepared.

We appreciate the opportunity to report on this project.

Sincerely yours,

John S. Gottschalk
John S. Gottschalk
Regional Director

EXHIBIT NO. 4

NEW YORK CENTRAL SYSTEM

C. E. DEFENDORF
CHIEF ENGINEER

466 LEXINGTON AVENUE
NEW YORK 17, N. Y.

July 11, 1961

SUBJECT: Flood Protection Project, Quaboag River
West Warren, Massachusetts

Division Engineer
New England Division
U. S. Corps of Engineers
424 Trapelo Road
Waltham 54, Mass.

Your File No. NEDGW

Attention: Mr. John Wm. Leslie
Chief, Engineering Division

Dear Sir:

Your letter of June 6, 1961, to our Mr. T. H. Scott at Springfield, Mass., relative to local flood protection project, Quaboag River, West Warren, Massachusetts, and enclosing a set of contract plans, has been forwarded to me for my comments and review.

Please be advised that the plans are satisfactory to the Railroad Company from a structural standpoint.

It is understood that you will provide a minimum of 12' from the cneterline of track to the top edge of riprap, as discussed with Mr. Scott.

Yours very truly,

C E Defendorf

CHIEF ENGINEER

EXHIBIT NO. 5

APPENDIX B

FLOOD LOSSES AND BENEFITS

PLATES

APPENDIX B

FLOOD LOSSES AND BENEFITS

1. DAMAGE SURVEYS

A damage survey was made in the flood area immediately after the 1955 flood and supplemented by a review in the Spring of 1959. These surveys consisted of detailed inspections of the industrial properties to ascertain the physical condition of the properties, the nature and amount of damages, depths of flooding, high water references and relationships between the 1955 flood and other flood stages. Damage data in the experienced flood furnished by owners and tenants were utilized, when in the judgment of the investigators the estimates were realistic. In other cases, owners' estimates were modified by investigators. Additional information was obtained from local and State officials.

Sufficient data were obtained to derive loss estimates for (1) the 1955 flood crest, (2) a stage 3 feet above the 1955 crest, and (3) intermediate stages below 1955 crest where marked changes in damage occur. The stage at which damage begins, referenced to the 1955 flood, was also determined.

2. LOSS CLASSIFICATION

Losses in the project area were principally industrial, with some loss to railroad and highway.

Primary losses have been classified as physical and non-physical. Physical losses comprise primary losses, such as damage to structures, machinery, and inventories, and the cost of cleanup and repairs. Non-physical losses include unrecoverable loss of business, wages or production, increased cost of operation and cost of temporary facilities.

Physical damage and a large part of the related non-physical losses were determined by direct inspection of property and evaluation of losses by property owners, tenants and field investigators.

3. RECURRING AND PREVENTABLE LOSSES

There has been no change in use in the flood area since the flood of 1955, with the exception of the one building adjacent to the river which has changed tenants. Estimates have been made of the recurring damages that would be experienced with various flood stages above and below the 1955 flood level. A recurrence of the 1955 stages would cause damages estimated at $1.1 million at 1961 price levels.

4. AVERAGE ANNUAL LOSSES

Estimated recurring losses were converted to average annual losses as a basis for determining annual benefits for use in economic evaluation. Estimated annual losses were derived by correlation of stage-damage, stage-discharge, and discharge frequency curves, to produce damage-frequency relationships in accordance with standard Corps of Engineers practice. Annual losses were derived only for those floods having a frequency of more than once in 400 years, inasmuch as the relative accuracy of discharge frequency, and stage-frequency relationships deteriorates sharply for the rarer floods. Annual losses for the flood area are estimated at $18,700 at 1961 prices.

Plates E-1 through E-7 show procedures used in converting recurring stage-damage data to curves of damage-frequency.

5. ANNUAL BENEFITS

a. Flood damage prevention benefits. Average annual flood damage prevention benefits were determined by deriving the difference between average annual losses under existing conditions and those losses remaining after construction of protective works. Annual losses resulting from floods having a frequency of more than once every 300 years will be eliminated by construction of a dike in the headwater portion of the project, and by improvement of the channel downstream. Average annual flood damage prevention benefits accruing to the protective works amount to $17,400.

b. Other tangible benefits. No increase in utilization of lands and buildings is expected to follow construction of the local protection project. Land available in the flood-free areas in the project vicinity have shown little development in the past few years. Few structural changes have been noted in this locality in more recent times. Construction of the project would relieve the threat of flooding and encourage the present industrial concerns to remain in the district.

c. Intangible benefits. Significant intangible benefits will be realized from construction of the proposed local protection project. The safety of the entire village of West Warren against fire is dependent upon the pumping facilities located in the project area. The assurance of non-interruption of employment for nearly 1,000 people will have a substantial effect on the economy of West Warren and surrounding communities. In addition, construction of the project would greatly lessen the flood threat to life and the potential danger of disease inherent in all floods.

NO. 3128. LOGARITHMIC PROBABILITY. DESIGNED BY HAZEN, WHIPPLE & FULLER. CODEX BOOK COMPANY, INC. NORWOOD, MASSACHUSETTS. PRINTED IN U.S.A.

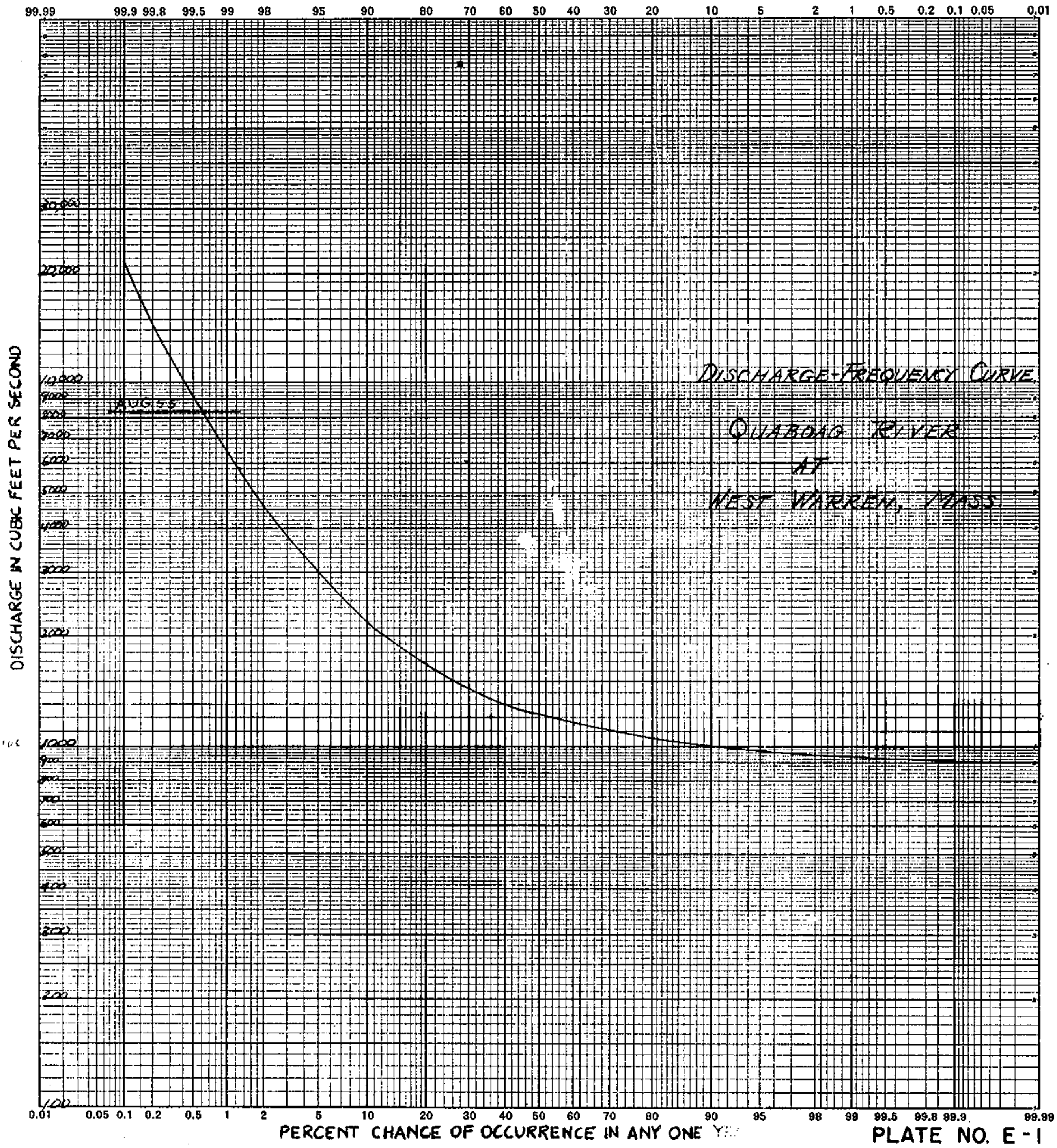
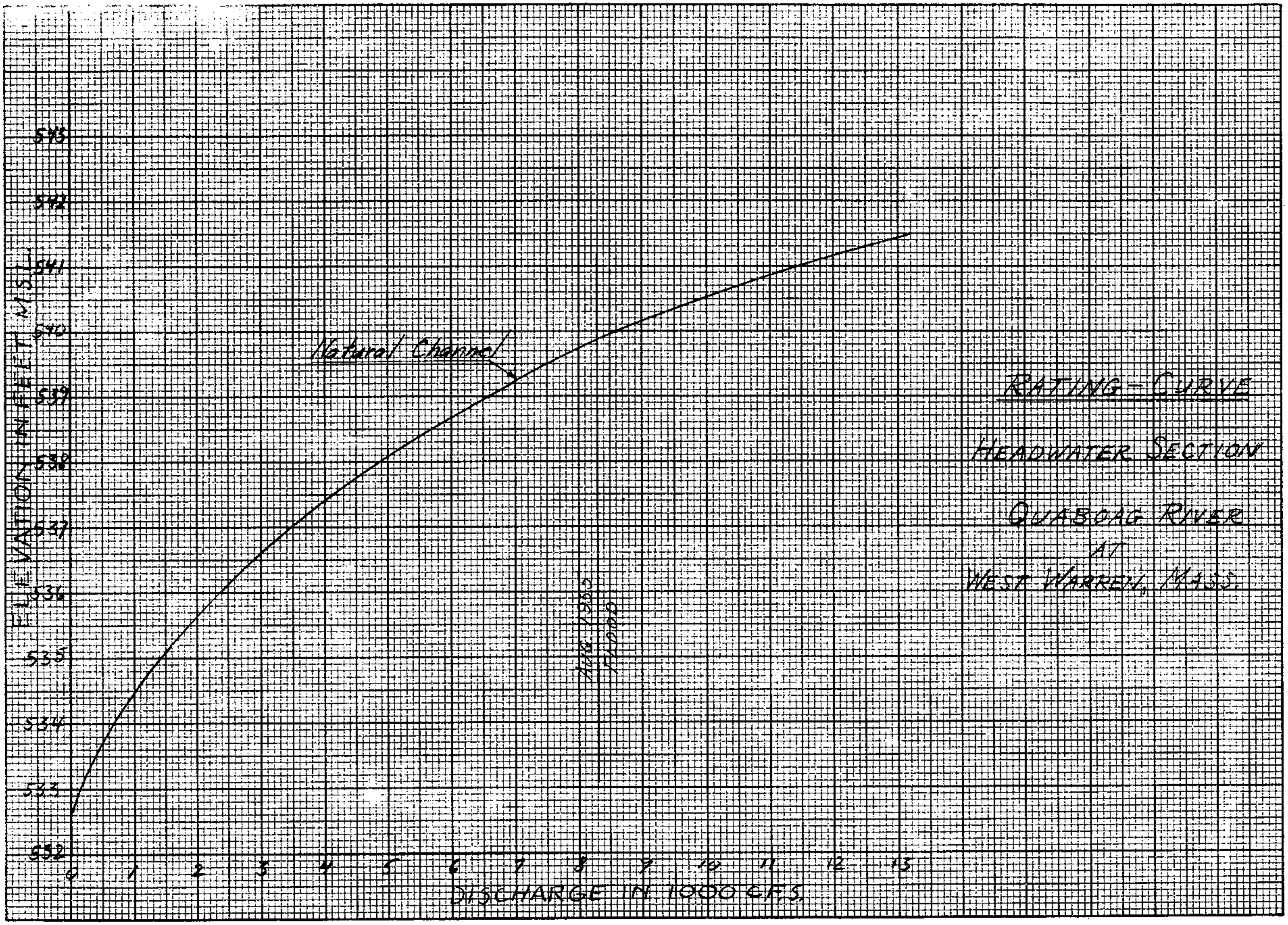
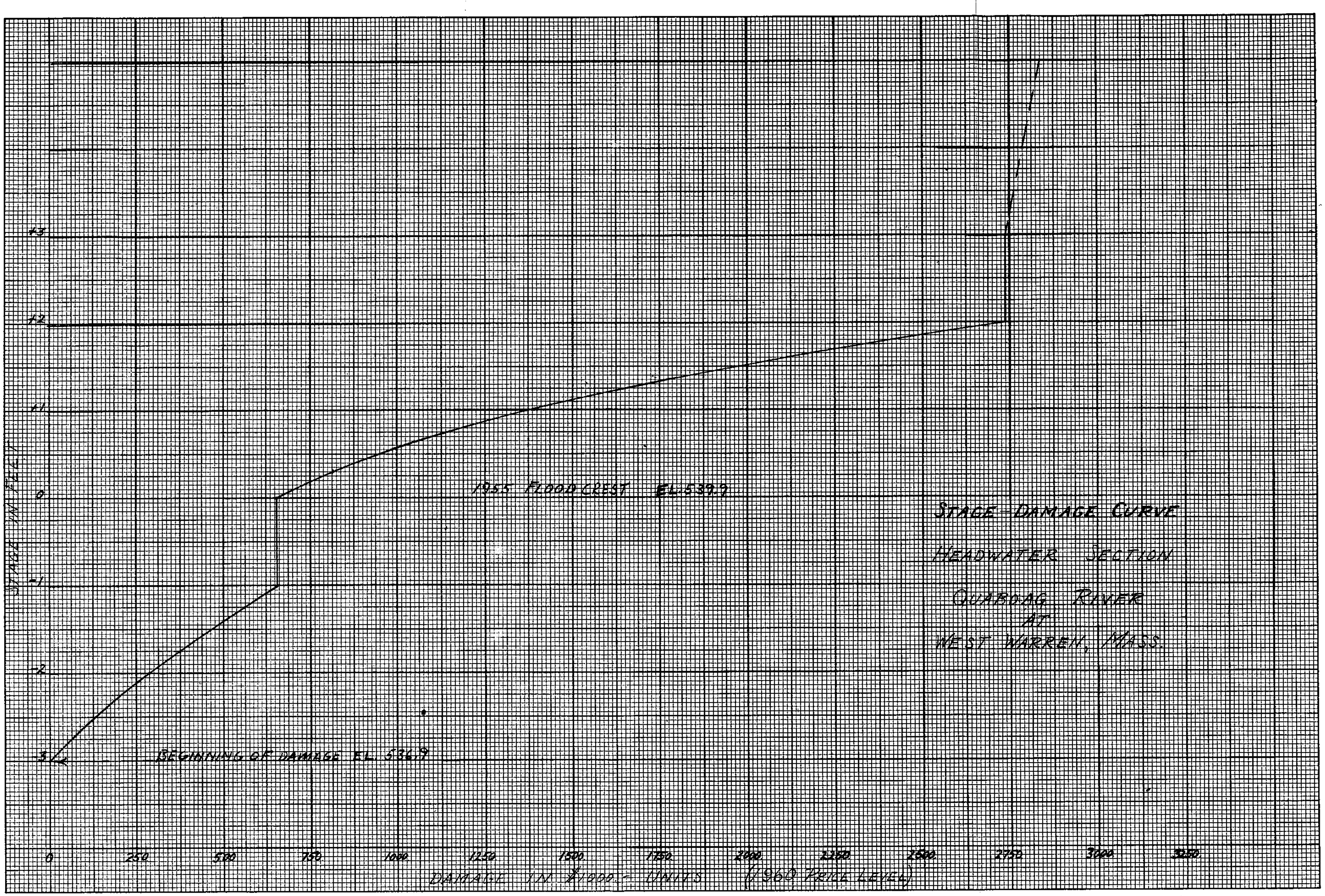


PLATE NO. E-1



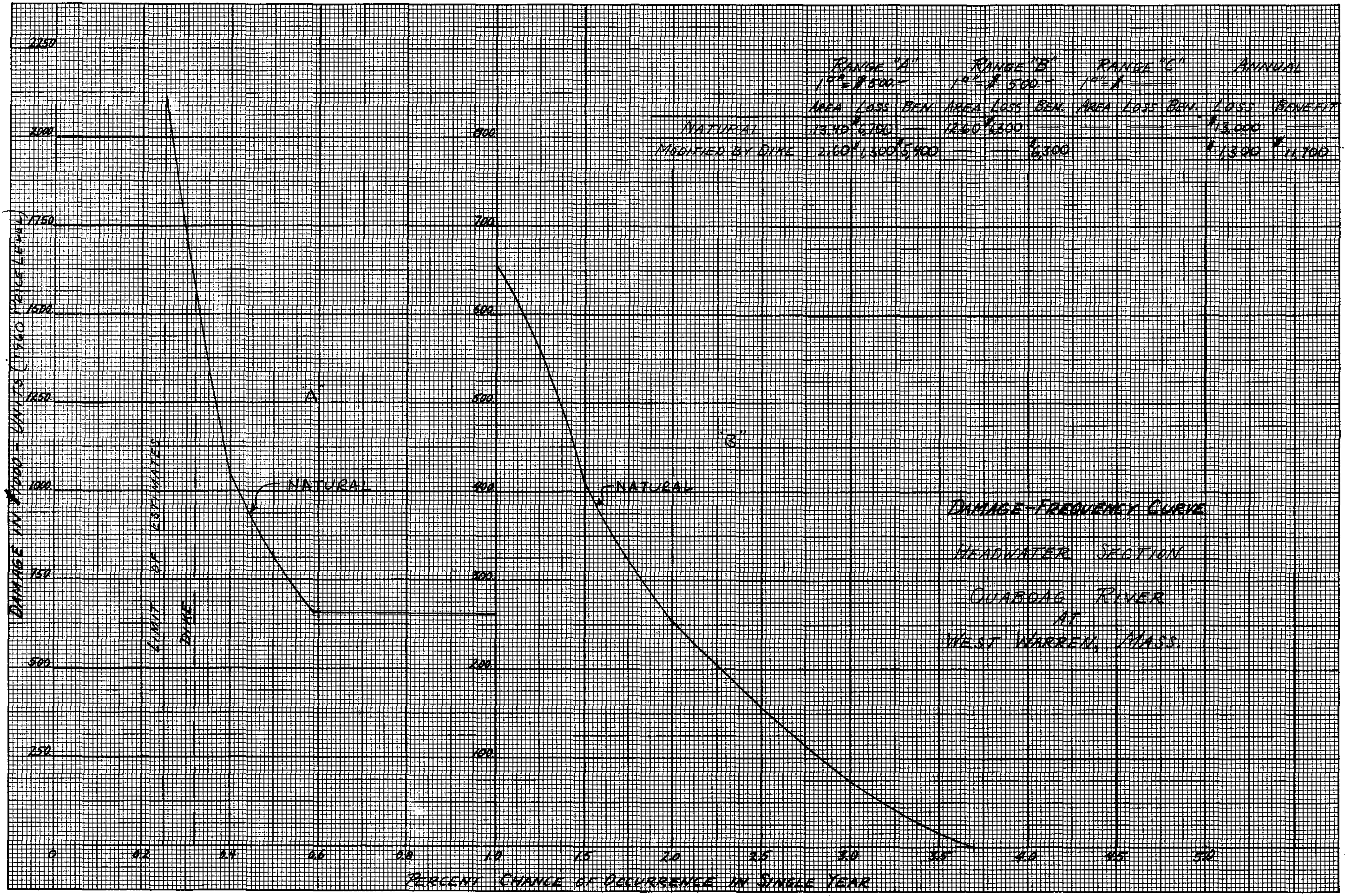
RATING CURVE
HEADWATER SECTION
QUABOAG RIVER
AT
WEST WARREN, MASS.
Natural Channel
AUG. 1955
FLOOD
ELEVATION IN FEET MSL
DISCHARGE IN 1000 CFS.
543
542
541
540
539
538
537
536
535
534
533
532
0
1
2
3
4
5
6
7
8
9
10
11
12
13

PLATE NO. E-2



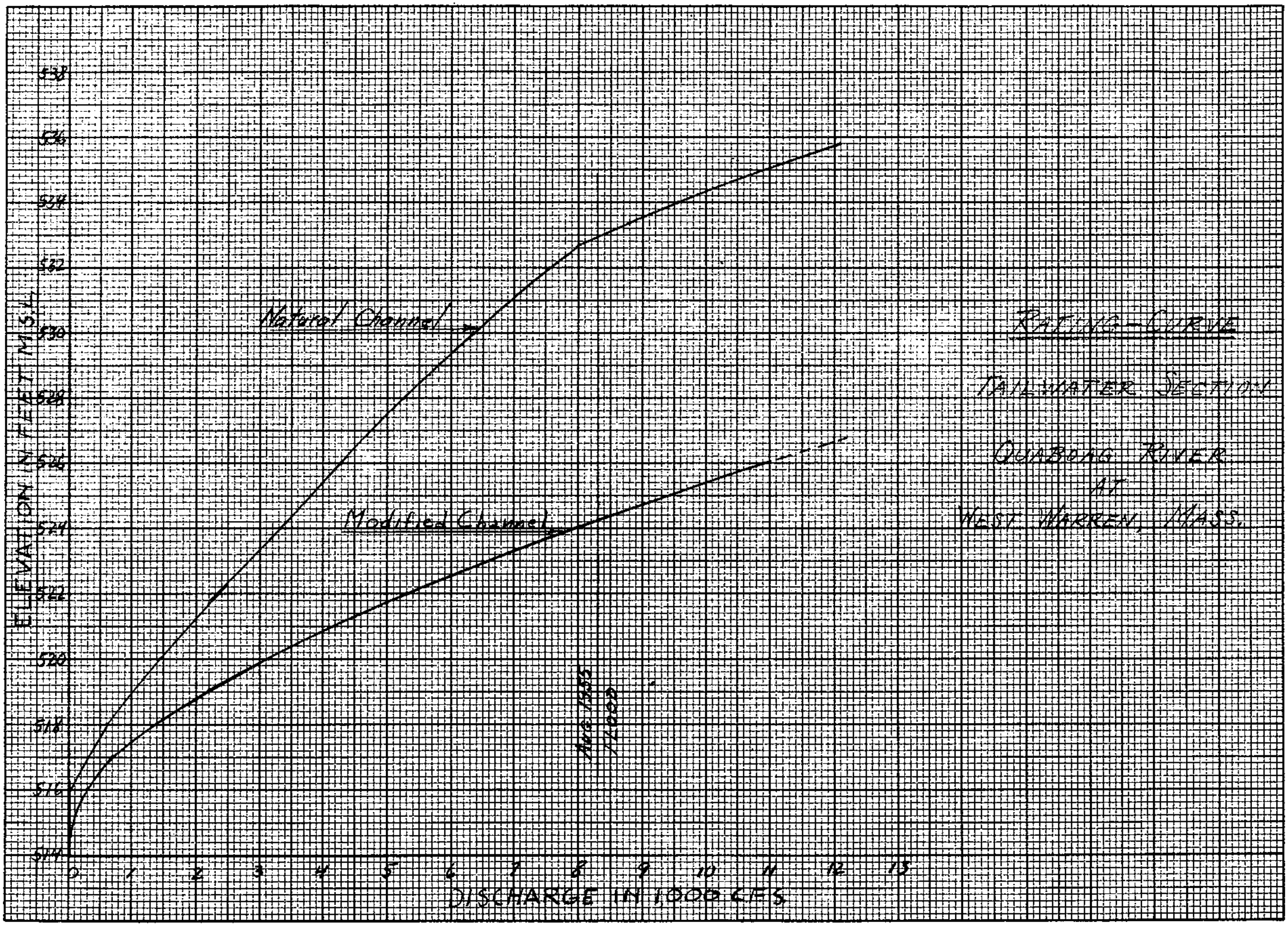
STAGE-DAMAGE CURVE
HEADWATER SECTION
QUABOAG RIVER
AT
WEST WARREN, MASS.
1955 FLOOD CREST EL 539.9
BEGINNING OF DAMAGE EL 536.9
STAGE IN FEET
+3
+2
+1
0
-1
-2
-3
0
250
500
750
1000
1250
1500
1750
2000
2250
2500
2750
3000
3250
DAMAGE IN $1000 = UNITS (1960 PRICE LEVEL)

PLATE NO. E-3



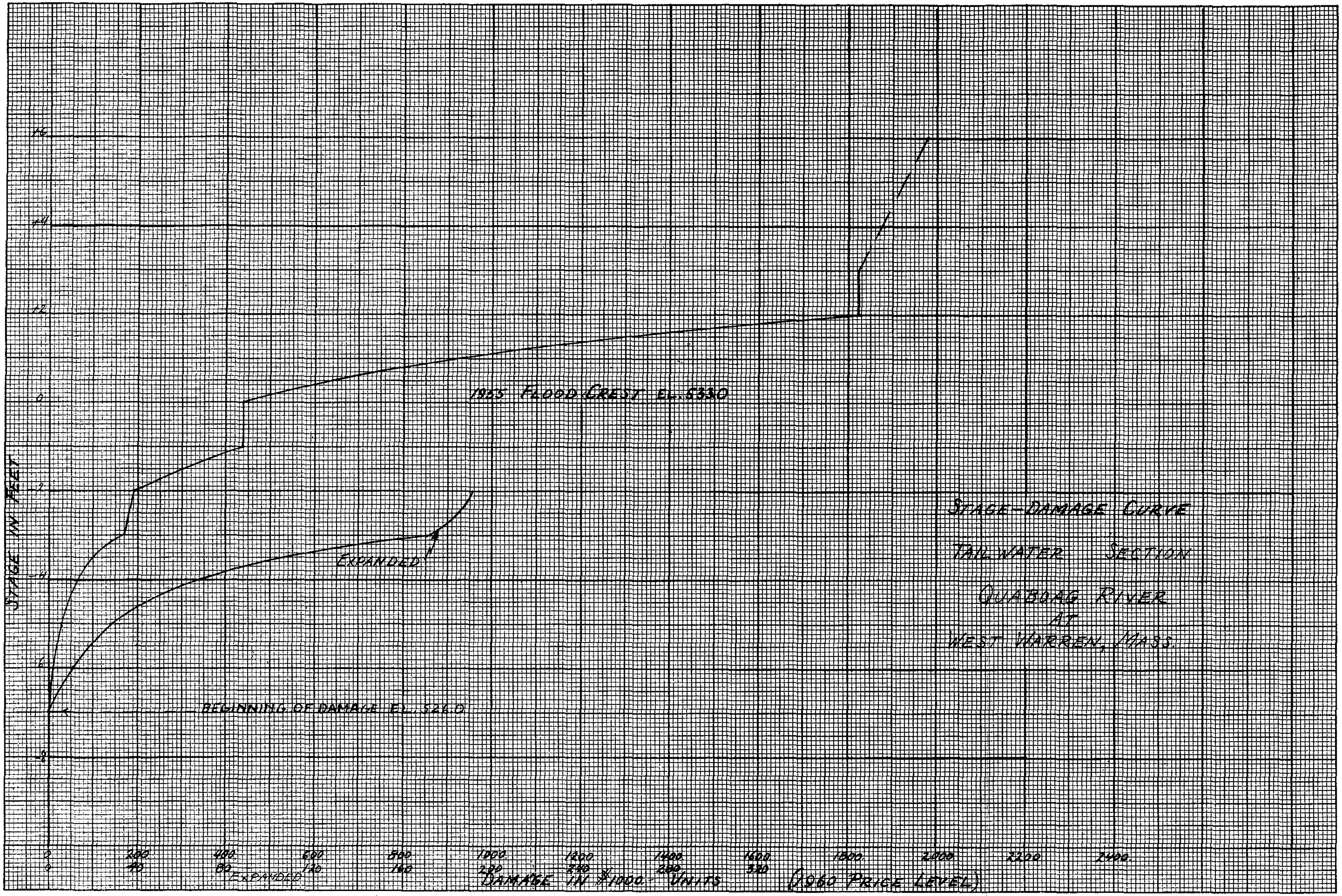
	RANGE "A" 1□" = $500.–			RANGE "B" 1□" = $500.–			RANGE "C" 1□" = $ ———			ANNUAL	
	AREA	LOSS	BEN.	AREA	LOSS	BEN.	AREA	LOSS	BEN.	LOSS	BENEFIT
NATURAL	13.40	$6,700	—	12.60	$6,300	—	—	—	—	$13,000	—
MODIFIED BY DIKE	2.60	$1,300	$5,400	—	—	$6,300	—			$1,300	$11,700

PLATE NO. E-4



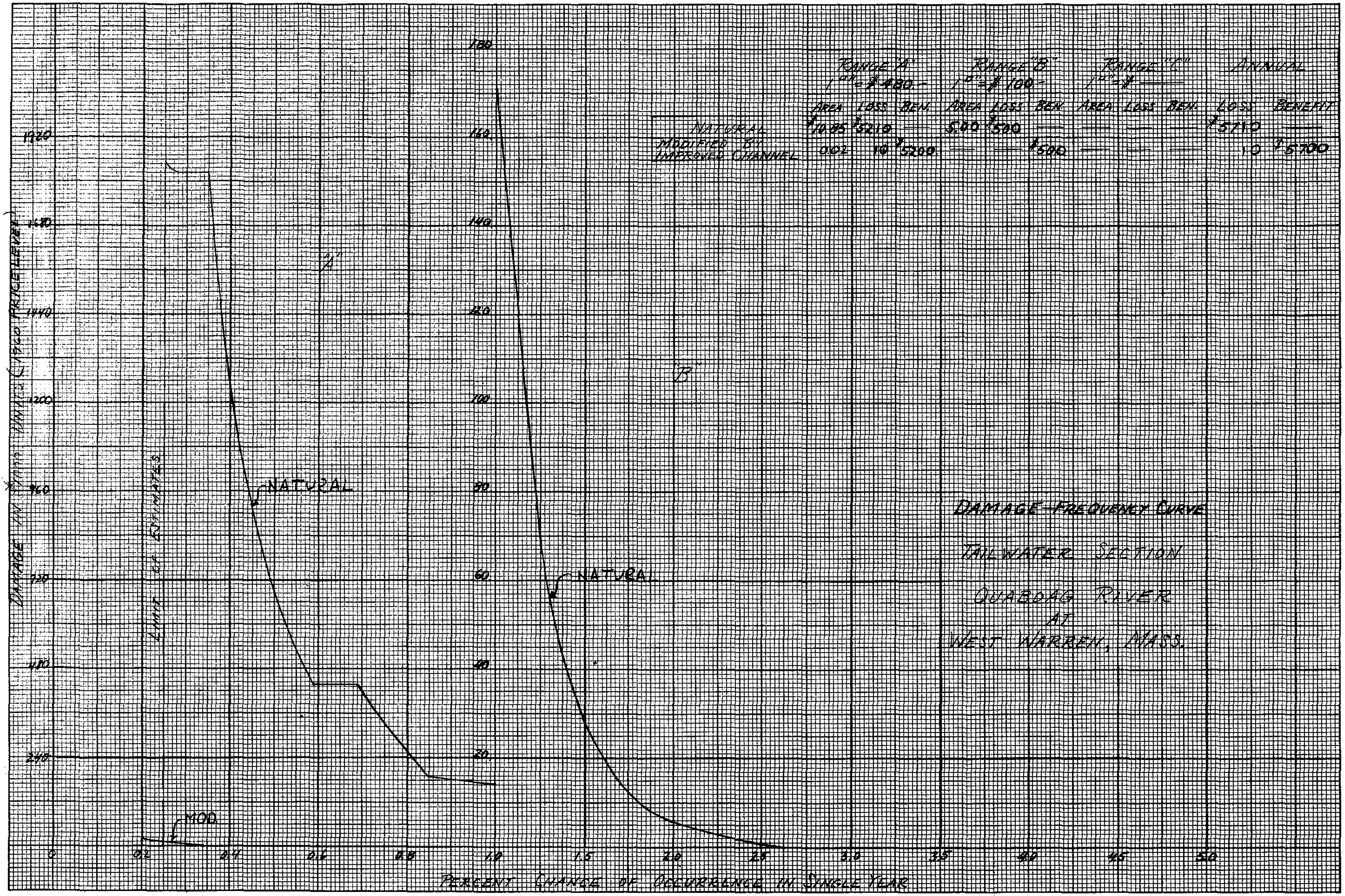
RATING-CURVE
TAILWATER SECTION
QUABOAG RIVER
AT
WEST WARREN, MASS.
Natural Channel
Modified Channel
Aug 1955
12,000
ELEVATION IN FEET M.S.L.
DISCHARGE IN 1000 CFS
514
516
518
520
522
524
526
528
530
532
534
536
538
0
1
2
3
4
5
6
7
8
9
10
11
12
13

PLATE NO. E-5



STAGE IN FEET
16
14
12
1955 FLOOD CREST EL. 533.0
EXPANDED
BEGINNING OF DAMAGE EL. 526.0
STAGE-DAMAGE CURVE
TAILWATER SECTION
QUABOAG RIVER
AT
WEST WARREN, MASS.
0 200 400 600 800 1000 1200 1400 1600 1800 2000 2200 2400
0 40 80 120 160 200 240 280 320
EXPANDED
DAMAGE IN $1000- UNITS
(1960 PRICE LEVEL)

PLATE NO. E-6



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	AREA	LOSS	BEN.	AREA	LOSS	BEN.	AREA	LOSS	BEN.	LOSS	BENEFIT
NATURAL	$10.85	$5210	—	5.00	$500	—	—	—	—	$5710	—
MODIFIED BY IMPROVED CHANNEL	0.02	10	$5200	—	—	$500	—	—	—	10	$5700

PLATE NO. E-7

APPENDIX C

LETTERS OF CONCURRENCE AND COMMENT

Exhibit No.	Agency	Letter Dated
1	Town of Warren, Massachusetts	June 6, 1961
2	The Commonwealth of Massachusetts	July 19, 1961
3	U.S. Department of Agriculture Soil Conservation Service	June 14, 1961
4	U.S. Department of the Interior Fish and Wildlife Service	July 7, 1961
5	New York Central System	July 11, 1961

TOWN OF WARREN
BOARD OF SELECTMEN
TOWN HALL

June 6, 1961

Steven A. Korzec, Chairman
Alice L. Leach, Clerk
Robert W. Williams

Karl F. Eklund,
Colonel, Corps of Engineers,
Deputy Division Engineer,
U.S. Army Engineer Div., N.E.
424 Trapelo Road, Waltham 54, Mass.

Dear Colonel Eklund:

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Regarding proposed plans and construction, preliminary plans being on file in our office. As per your request, it is the opinion of the present Board of Selectmen, Town of Warren, that local interests are willing and able to comply with the requirements specified under the authority of P.L. 685, 84th Congress, adopted 11 July 1956.

This is not a legal commitment at this time, but we wish to state our willingness to extend our utmost cooperation.

Thanking you, we remain

Sincerely yours,
Steven A. Korzec
Board of Selectmen,
Steven A. Korzec Chairman.

SAK-R

EXHIBIT NO.1

THE COMMONWEALTH OF MASSACHUSETTS
EXECUTIVE DEPARTMENT
STATE HOUSE, BOSTON

JOHN A. VOLPE
GOVERNOR

July 19, 1961

Brigadier General Seymour A. Potter, Jr.
Division Engineer - New England Division
U. S. Corps of Engineers
424 Trapelo Road
Waltham, Massachusetts

Dear General Potter:

I am in receipt of your letter of June 9, 1961, in regard to the proposed project to provide local flood protection works along the Quaboag River in the West Warren section of the Town of Warren, Massachusetts. This project is to be constructed under Public Law 685-84th Congress, adopted in July 1956.

In your letter you point out:

1. That the estimated Federal cost of the project is $280,000.

2. That the non-Federal cost for acquiring land and for providing funds for features of the project other than flood control will be $45,000 and that the Town of Warren has given its assurance that it will contribute that sum.

The question that is raised in your letter is whether the Commonwealth of Massachusetts will assume full responsibility for all project costs in excess of the Federal cost limitation of $400,000 in the event that the cost of the final project should exceed that amount. The Commonwealth of Massachusetts has enacted

EXHIBIT NO 2

a law, Chapter 763 of the Acts of 1957, including the appropriation of the necessary funds to pay the excess cost of flood control projects constructed under this Federal flood control program.

The sums appropriated in Chapter 763, however, revert to the General Fund on July 1, 1962. As it will be impossible to know before that date whether there will be any excess in connection with the West Warren project, I am suggesting to the Division of Waterways in the Massachusetts Department of Public Works that it request that the present appropriation be continued. We consider this project an essential one for providing flood protection in the Town of Warren and will gladly assume the responsibility for any cost of construction in excess of $400,000, providing the continuation of the existing appropriation is authorized by the Legislature when it reconvenes in January.

I trust that the Secretary of the Army will authorize the allotment of the necessary appropriation for this project.

Sincerely,

John A. Volpe

Governor

UNITED STATES DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

29 Cottage Street, Amherst, Massachusetts

June 14, 1961

General Seymour A. Potter, Jr.
Division Engineer, New England
Corps of Engineers
424 Trapelo Road
Waltham 54, Massachusetts

Dear General Potter:

We have reviewed the proposed plan of protection for a local flood protection project along the Quaboag River at West Warren, Massachusetts.

We believe that this local protection project as described in your letter of June 9, 1961, is needed as part of the over-all plan for the Upper Quaboag River Watershed. As you know, the flood prevention and watershed protection work plan for the Upper Quaboag River Watershed under P. L. 566, is based on the premise that the local protection project at West Warren will be installed.

Sincerely yours,

Benj. Isgur

Benjamin Isgur,
State Conservationist.

EXHIBIT NO. 3

ADDRESS ONLY THE
REGIONAL DIRECTOR

UNITED STATES
DEPARTMENT OF THE INTERIOR
FISH AND WILDLIFE SERVICE
BUREAU OF SPORT FISHERIES AND WILDLIFE
59 TEMPLE PLACE
BOSTON, MASSACHUSETTS

NORTHEAST REGION
(REGION 5)
MAINE
NEW HAMPSHIRE
NEW YORK
VERMONT
PENNSYLVANIA
MASSACHUSETTS
NEW JERSEY
RHODE ISLAND
DELAWARE
CONNECTICUT
WEST VIRGINIA

July 7, 1961

Division Engineer
New England Division
U. S. Army Corps of Engineers
424 Trapelo Road
Waltham 54, Massachusetts

Dear Sir:

Reference is made to your letter of June 9, 1961 regarding a proposed local protection project along the Quaboag River at West Warren, Massachusetts, and in which you request comments from this Bureau regarding the effects of the project on the fish and wildlife resources. This letter constitutes our conservation and development report and has the concurrence of the Massachusetts Division of Fisheries and Game.

It is our understanding that the proposed plan of protection includes a dike and flood wall upstream of the existing dam; the construction of concrete buttress walls to protect Buildings No. 7 and 11; the removal of the existing stone arch bridge; the removal and replacement of an oil pipeline bridge by a new bridge with adequate waterway; channel excavation in the Quaboag River below the existing dam; rock slope protection along sides and bottom of the river; and the reconstruction of the north pier footing of the overhead South Street Bridge to permit channel deepening at this location.

We have considered the impact of this project and conclude that it will have no significant effect on fish or wildlife resources.

We would appreciate being advised if any major changes are made in the project plan, in order that a new report may be prepared.

We appreciate the opportunity to report on this project.

Sincerely yours,

John S. Gottschalk
Regional Director

EXHIBIT NO. 4

NEW YORK CENTRAL SYSTEM

C. E. DEFENDORF
CHIEF ENGINEER

466 LEXINGTON AVENUE
NEW YORK 17, N. Y.

July 11, 1961

SUBJECT: Flood Protection Project, Quaboag River
West Warren, Massachusetts

Division Engineer
New England Division
U. S. Corps of Engineers
424 Trapelo Road
Waltham 54, Mass.

Your File No. NEDGW

Attention: Mr. John Wm. Leslie
Chief, Engineering Division

Dear Sir:

Your letter of June 6, 1961, to our Mr. T. M. Scott at Springfield, Mass., relative to local flood protection project, Quaboag River, West Warren, Massachusetts, and enclosing a set of contract plans, has been forwarded to me for my comments and review.

Please be advised that the plans are satisfactory to the Railroad Company from a structural standpoint.

It is understood that you will provide a minimum of 12' from the cneterline of track to the top edge of riprap, as discussed with Mr. Scott.

Yours very truly,

C E Defendorf

CHIEF ENGINEER

EXHIBIT NO. 5